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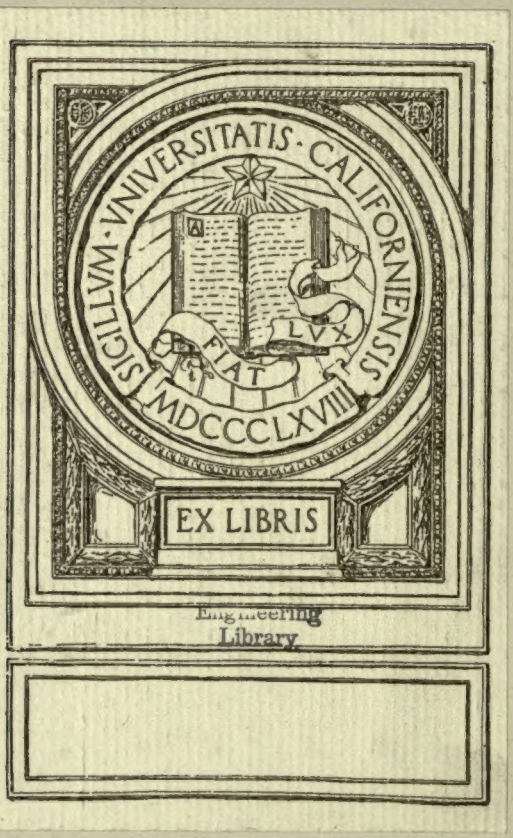

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A COURSE

in the

ELEMENTS OF SURVEYING

by

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Prepared Especially for the

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Course 1A-1B **Elements of Surveying** Swafford

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PLAN OF THE COURSE

This course in Plane Surveying, offered by the Extension Division of the University of California comprises forty (40) assignments, each treating some definite part of the subject, and aiming in their scope to present the work embraced in the regular surveying courses, Civil Engineering 1A, 1B, and C. E. 3, College of Civil Engineering, University of California; excepting, that in the Extension course the problems for field solution and practice with instruments have been specially selected and arranged for your work without the immediate supervision of an instructor.

The problem work, therefore, calls for special care on your part. You should give particular heed to (1st) the purpose; (2nd) the method; (3rd) the care; refinements, and precision of the work; (4th) the record of field notes - sketches, illustrations, diagrams, etc., and (5th) the result - the lesson learned or the training gained.

The plan of this course requires that you shall study each assignment in acareful, painstaking way; not merely reading it over, nor even committing it to memory, but making a full and critical study of each part, so as to gain a clear understanding of the thought content, to see its bearing on the subject of surveying as a whole, and thus to acquire step by step the science of surveying.

References are given in each assignment to standard texts on the subject of surveying, including Tracy, Johnson, Breed & Hosmer, and Raymond. Of course it is not likely that you will have

access to all or even several of these works, but it is hoped that you may have one or more of them, and also a manual of surveying instruments, such as published by manufacturers descriptive of their goods. These manuals contain much information, tables, hints, etc., of value to the engineer. Attention is called here to those issued to the engineering profession by Bausch and Lomb, Rochester, New York, C. L. Berger and Sons, Boston, Mass., W. and L. E. Gurley, Troy, New York, and others, which may be procured at small cost by addressing the firms named above. The General Land Office, Washington, D. C., also has in preparation a Manual on the Survey of the Public Lands, which may be had for a small fee from the Bureau of Publication, Washington, D. C. Referenoes will frequently be made to the foregoing works, and their careful perusal is earnestly urged.

Accompanying each assignment are questions to be answered, problems to be solved, and field exercises to be worked out. Your papers are to be sent to the Extension Division for correction, criticism, and grading. Your standing in the course is determined by the paper you submit; your progress will depend largely upon your ability to profit by corrections and criticisms.

Each paper submitted by you should be worked out carefully and punctiliously, special care being taken to place everything, from heading to close, in a neat, orderly arrangement, with any field notes, tabulations, or sketches to accompany the same. Field notes and sketches (or illustrations) should preferably be done in pencil. Use a 4H pencil and letter (not write) the notes in approved styles.

(See illustrative, specimen pages of Field Notes displayed in the assignments from time to time.)

The required computations must be made in good arrangement, all operations clearly stated, and conclusions or answers properly indicated. The actual work of multiplication, division, extraction of roots, etc., need not be returned with the solutions, but the processes and their results must be clearly shown. Logarithmic computations should give the logarithms used. Where slide rules or other mathematical devices are employed, the fact should be definitely stated. Where tables are used, the title and source should likewise be mentioned. This of course does not apply to tables of logarithms, powers and roots, etc.

In all your work bear in mind that the purpose is to acquire a knowledge, not only of the subject of surveying as a theory, but of its methods and expressions as embodied in principles, problems, notes, maps, and instruments employed in field and office. Without such knowledge and the training that goes with it, hand in hand, this course will be of small avail in giving you the real training intended.

INTRODUCTION

Surveying, like most other work in applied science, is an art. The strict application of formulas is not always possible. In many problems special devices must be used, the correct solution of a problem must be set aside and approximations, more or less crude, must be substituted to meet the rights of legal ownership under the laws of the land. Errors in previous surveys of the same lines or areas must be considered and corrected or the new survey must conform to the actual conditions met with on the ground in spite of written evidence to the contrary.

To accomplish such things, then, the surveyor must be a man at once honest, sincere, dependable, energetic, ingenious, and observant. He must be patient in his work and with other people. He must be ready to give an unbiased opinion as to the rights of disputants when called upon to do so, and should always be sure of the correctness of his work or its limitations before submitting results to his employer or to other persons. He must have a thorough knowledge of the fundamental principles of surveying such as will enable him to solve all of his problems correctly.

In these assignments it is assumed that you have a good understanding of the common processes of arithmetic, algebra, plane geometry, plane trigonometry, and the use of mechanical drawing instruments. It should be your endeavor to acquire a knowledge of solid geometry, spherical trigonometry, and physics, as of real value to you in your studies and in later work in the field.

It is further assumed that you will have access to the common equipment of the surveyor so that the explanations of the various instruments and the methods of their adjustment and use can be understood. Lacking such equipment it will be hard for you to grasp the significance of much of the course and your studies will result in a training in geometry and not in a training in surveying methods.

You are cautioned not to jump to the common and erroneous conclusion that what is said in this series of assignments is all that can be said on the subject of surveying. It is not intended to give you skill as a surveyor, but to give a good, general training in the fundamental principles underlying the methods of the surveyor in the field and office, coupled with a brief discussion of the common problems met in practice. Skill, in any line of endeavor, comes only through long practice.

The real student will find suggestions that will lead him far into the field of surveying.

DEFINITIONS

FOREWORD:

In this assignment it is intended to give a general outline of the purpose of surveying and some of the common conceptions, assumptions, and definitions necessary to a proper understanding of the more detailed problems which are to follow.

(1) SURVEYING

The process of measuring lengths of lines, magnitudes of angles, and differences of elevation at or near the surface of the earth for the purpose of establishing boundaries, establishing elevations of points, map-making, and construction, is called surveying.

(2) THE EARTH

The earth is approximately a sphere. The diameter from pole to pole being 26.87 miles shorter than a diameter in the plane of the equator, the equatorial diameter, as computed by Clarke in 1866 and adopted by the U. S. Coast and Geodetic Survey, is 41,852,124 feet, while the corresponding polar diameter is 41,710,242 feet.

(3) SEA-LEVEL

The fundamental datum for elevation of points on or near the surface of the earth is taken at sea level. The layer of water covering the greater portion of the earth has nearly the same form as the mean globe. A slight variation is due to the action of the tides and the rotation of the earth on its axis. Since there is a rise and fall of tide twice a day for any one locality, it has been

decided that the MEAN LEVEL of the ocean will be called sea-level. In all the common determinations of elevations sea-level is said to have a zero elevation.

A point well worth remembering is that most of the problems in surveying occur above sea-level.

(4) MERIDIANS

Imaginary lines passing completely around the earth and forming a closed curve two points of which lie in the north and south poles of the globe are called meridians. To illustrate--

Imagine that a person starts from a point on the surface of the earth and travels due north until the north pole is reached. Continuing through the pole in the same line, not deviating from it in any way, his course will be due south until he reaches the south pole; thence he travels due north to his original position. The path traversed is a meridian.

The geometric and astronomical definition is as follows:

A meridian is the line on the surface of the earth cut by a plane passing through the north and south poles of the earth.

(5) PARALLELS OF LATITUDE

Parallels of latitude are imaginary circles on the surface of the earth lying in planes parallel to the plane of the equator and having their centers in the line joining the poles (polar axis) of the earth.

(6) LONGITUDE

The angular distance from an agreed fundamental meridian to

the meridian through the point in question, measured in the plane of the parallel of latitude through the same point, is the longitude. The fundamental (prime) meridian for English-speaking countries is through Greenwich, England. Longitudes are measured East and West from Greenwich, in each direction from 0° to 180°.

It is to be noted that longitude is the angular measure so that while meridians converge toward the pole and the linear distance between them gets less as the pole is approached, the longitude of the meridian in question does not change so long as neither pole is passed.

(7) LATITUDE

Latitude is the angular distance from the Equator north or south to the parallel of latitude through the point in question, measured in the plane of the meridian through the point. Latitudes range from 0 degrees at the Equator to 90 degrees at the poles.

(8) A PLUMB LINE

The line determined by the position of a cord at the lower end of which is fastened a weight sufficient to stretch the cord into a straight line, the cord then being suspended in quiet air and allowed to come to rest, is called a plumb line.

A common definition of a plumb line is about as follows:

A plumb line is a line pointing toward the center of the earth, i. e., a radius of the earth prolonged. This, however, is not strictly true owing to the rotation of the earth and the irregular distribution of mass within the earth. In the northern hemisphere, generally, the plumb lines point slightly south of the

geometric center of the earth.

No two plumb lines are parallel.

A plumb line points to what is known as the ZENITH or the point in the heavens vertically overhead at any point on the earth's surface. The NADIR is a corresponding point found by prolonging the plumb line through the earth to a diametrically opposite point on the celestial sphere.

(9) A LEVEL SURFACE

A surface that is everywhere perpendicular to the plumb line is a level surface. Thus, it is seen that a level surface is not a flat surface or a plane, but a curved surface conforming, approximately, to the general shape of the earth.

(10) A LEVEL LINE

A level line, is then, a line in this curved surface and is, itself, curved. What is ordinarily called a level line is a geometrically straight line tangent to a level line at which point of tangency the straight line is perpendicular to the plumb line. A level line, on the other hand, is perpendicular to any plumb line that may intersect it.

(11) A STRAIGHT LINE

A straight line on the surface of the earth is really a portion of a great circle of the earth, e. g., a meridian is a straight line. A geometrically straight line is one that would be tangent to a sphere if drawn perpendicular to a plumb line, or would pass through the surface of the sphere in two points or would not touch the sphere at all - all three cases are possible.

A geometrically straight line would possibly cut through the surface of the earth in more than two places owing to inequalities of the surface.

(12) GEODETIC SURVEYING

The branch of surveying which, owing to the extent of the surveys, takes into consideration the spheroidal form of the earth is termed Geodetic Surveying. That is, in the survey of a continent or state the lines are so long that they are appreciably curved and the angles of the various polygons making up the survey are angles of spherical polygons. This type of surveying is very difficult, requiring men of special training and experience in the prosecution of both field and office work.

(13) PLANE SURVEYING

The branch of surveying which, owing to the limited extent of the surveys, does not consider the spheroidal form of the earth, is known as plane surveying. In other words the assumptions made are:

1. A level surface is a flat surface.
2. A level line is a straight line.
3. A "straight line" on the earth is geometrically straight.
4. All plumb lines within the limits of the survey are parallel.
5. All angles of a polygon are plane angles.

This branch, the more common and elementary of the two types, will constitute the greater part of the work of this course.

(14) NATURE OF MEASUREMENTS IN PLANE SURVEYING

Surveying, as before stated, is for the purpose of determining

the relative positions of points and lines on or near the surface of the earth. A moment's reflection will bring you to the realization that in the use of the common equipment of the surveyor the only things possible to determine are the positions of points, the distances (both horizontal and vertical) between them, and the angles between lines and planes. In determining the shape of a field, a building, or a cross-section in question the surveyor merely takes a sufficient number of measurements to enable him to establish, geometrically, the outline of the thing measured.

Where possible, it may be best to take direct measurements of the line or angle under consideration. But direct measurements may involve physical difficulty and lack of what is called "reasonable accuracy". For example, it may be possible to tape the length of a line across a swamp or a line across hill and valley for a long distance, but the error in the work and the physical exertion will often be far in excess of what is reasonable. It is usually possible to find some device by which such a determination may be made indirectly so that the result will be more accurate and the work of measurement far less troublesome and exhausting.

(15) ACCURACY VERSUS COST

Nearly all work in surveying can be done very accurately, but it is neither necessary or advisable to attempt to maintain a high degree of accuracy in all problems, nor to make all measurements in different parts of the same problems equally accurate. If a survey is being made to find the acreage of a field planted to a certain crop and the area is to be determined to the nearest one-tenth

the nearest ten or fifteen minutes and the lengths of the courses to the nearest link (7.92 inches). A later survey is made of the property, but the value of the land has increased to $200 per acre, in which case the surveyor should measure the boundaries of this field with the transit and steel tape determining the angles to the nearest minute and the distances to the nearest one-tenth of a foot. It is not to be understood that this accuracy is in strict ratio to the increase in value of the property nor should it be assumed that the new measurements will check the old but roughly. If the original surveyor executed his work properly and, though his measurements were more or less in error, set proper monuments which can be found intact by the later surveyor, the new survey will be a survey of the same field. Unfortunately, however, the earlier surveyors did not consider the possibility of a later survey or were apt to be careless about such things; consequently much labor and expense is the lot of the surveyor who makes the more accurate survey. It is seen that a large question of rights and what constitutes the true position of a boundary line is opened up by such problems.

(16) KINDS OF MEASUREMENTS MADE IN PLANE SURVEYING

When the area of a field is given as 160 acres the area meant is that of the polygon formed by projecting the boundaries of the field onto a horizontal plane. It may be that the boundaries of the field are on a hillside and do not lie in horizontal planes, but experience has shown that it would be difficult and impracticable to consider the lengths of the lines that would be found by following

the slope of the ground. It is seen, then, that what is called the length of a line in surveying is the horizontal projection of the line (This rule holds unless the length is qualified by some term such as "slope length", etc.). Usually the measurement of a line is made so that the horizontal projection is obtained directly.

In a similar manner the angles of a polygon on the ground are given as the angles between the horizontal projections of the lines.

In problems where the difference in elevation of points is important the measurements of such quantities are usually made so as to give, directly, the vertical distance.

There are numerous cases arising where the distance between two points will be measured on a slope. In such cases the vertical angle must be determined or the vertical difference in elevation between the extremities of the line must be found to make it possible to compute the horizontal distance. Where the slope distance and the vertical angle, or where the slope distance and horizontal distance are given, it will be possible to find the difference in elevation. When such a slope measurement is taken, it is subsequently reduced to a horizontal distance and to a vertical difference in elevation.

To summarize:

1. Distances are usually horizontal.
2. Differences in elevation are usually vertical.
3. Angles are measured in a horizontal plane or a vertical plane.

4. If measurements of any linear or angular quantities are are made in other ways, they are usually reduced to vertical and horizontal quantities.

PROBLEMS

1. Show by a diagram, carefully constructed, that no two plumb-lines are parallel.

2. Show that a level line between two points on the earth's surface is longer than a straight line joining the same points.

3. From the values of equatorial and polar diameters given on page 8, Assignment 1, compute the length of one minute of arc of longitude, and the mean length of one minute of arc of latitude.

4. Determine which is greater, the superficial area of a sloping field or the acreage as measured by the usual methods of plane surveying. Give reasons for following the methods of surveying land areas.

You will confer a favor upon the instructor and upon other students if you point out anything in the assignments of this course which seems to be obscure. Write questions to the instructor, if you care to do so, when you send in your papers.

UNIVERSITY OF CALIFORNIA EXTENSION DIVISION
CORRESPONDENCE-STUDY COURSES IN TECHNICAL SUBJECTS

Course 1A Elements of Surveying Swafford

Assignment 2

LINEAR MEASUREMENTS

FOREWORD:

The following discussion treats of the measurements of linear distances, particularly of those along the surface of the earth, or what are generally designated as horizontal measurements. Various types of measuring devices will be illustrated and described and particular emphasis laid upon the more common methods of measurement.

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(17) A LINEAR DISTANCE

The distance or length in a straight line between two points, is called the linear distance between those points. The linear measure or magnitude of the distance is expressed in terms of some established or arbitrary unit, e. g., a linear distance of 9 feet may be expressed as 9 feet, 3 yards, 108 inches, or 3 2/5 paces, the unit used depending, largely, upon the nature of the problem and the custom of the person making the measurement.

(18) THE UNITS OF MEASURE

The units used in the United States by the surveyor are either of English or French origin. The English units have the following relations:

Linear Measure			Square Measure		
12 inches	= 1 foot		144 sq. inches	= 1 sq. foot	
3 feet	= 1 yard		9 sq. feet	= 1 sq. yard	
5 1/2 yards	= 1 rod		30 1/4 sq. yards	= 1 sq. rod	
16 1/2 feet	= 1 rod		160 sq. rods	= 1 acre	
320 rods	= 1 mile		43,560 sq. feet	= 1 acre	
5280 feet	= 1 mile		640 acres	= 1 sq. mile	

Volumes		
1728 cu. inches	= 1 cu. foot	
27 cu. feet	= 1 cu. yard.	

An important system of units, of English origin, used by the surveyor is based on the rod. A measuring device known as the Gunter's Chain is made just 4 rods or 66 feet long. This length is divided into 100 parts, called links, each link having a length of 7.92 inches, hence a table of measures may be written in terms of links and chains:

Linear Measure	Square Measure
10 links = 1 rod	100 sq. links = 1 sq. rod
100 links = 1 chain	10000 sq. links = 1 sq. chain
1 chain = 66 feet	1 sq. chain = 16 sq. rods.
80 chains = 1 mile	10 sq. chains = 1 acre
	6400 sq. chains = 1 sq. mile

French units of measure are based on the International Metre and are as follows:

Linear Measure

10 millimetres = 1 centimetre
10 centimetres = 1 decimetre
10 decimetres = 1 metre = 39.37 inches (English)
10 metres = 1 dekametre
10 dekametres = 1 hektometre
10 hektometres = 1 kilometre = 1000 metres

Square Measure	Volumes
100 sq. mm. = 1 sq. cm.	1000 cu. mm. = 1 cu. cm.
100 sq. cm. = 1 sq. dm.	1000 cu. cm. = 1 cu. dm.
100 sq. dm. = 1 sq. m.	1000 cu. dm. = 1 cu. m.
100 sq. m. = 1 are	
100 ares = 1 hectare	
100 hectares = 1 sq. kilometre	

In times past and in the present day the English system of units has been the more common in this country. The Gunter's chain was used in much of the land surveying done in the early history of nearly every part of the United States. The reason is obvious when one understands the simple relation of the chain to the mile, acre, and section. Consequently many deeds and records

of early surveys appear in chains. For this reason the surveyor should become thoroughly familiar with this system of units.

Later surveys, where land has become valuable, usually give the linear dimensions in feet and fractions thereof. In this connection it should be pointed out that the surveyor usually takes measurements of linear quantities in feet and decimal fractions of feet. In other words, a foot is divided into tenths, hundredths, etc., and a distance is given as 756.78 feet rather than as feet, inches, and fractions of inches. Again, the surveyor should be familiar with the relation of decimal fractions of a foot to inches so that he can quickly change measurements to ordinary units for mechanics and others not familiar with such fractions.

TABLE I

Decimals of a Foot in Inches.

Decimal of a foot		Inches
0.01	=	1/8-
0.04	=	1/2-
0.08	=	1-
0.17	=	2+
0.25	=	3 exact
0.33	=	4-
0.50	=	6 exact
0.75	=	9 exact

For rough approximations the above equivalents will be found convenient, though for precise work it will be necessary to carry the relation out to a finer degree of accuracy.

The French (or Metric) system is shown for the reason that many surveyors are constantly leaving this country for countries where such units are employed and because much of the work of the

U. S. Coast and Geodetic Survey and other departments of the United States Government is carried out in metric units.

(19) MEASURING DEVICES

The commonest measuring device is the ordinary foot or two-foot rule. A finely constructed rule or scale of one foot in length is used by the engineer or surveyor. The graduations are given, usually, in decimal fractions of inches. This rule is used chiefly for office work in making scale drawings, but it will often have a proper place in precise measurements in the field. The common type of folding carpenter's rule is well known to almost everyone interested in the mechanical arts. This rule is graduated variously most commonly in inches and sixteenths of an inch.

The yard stick and ten-foot pole are used in short, rough measurements. Formerly the ten-foot pole was used in certain types of surveying problems.

The cloth tape, made of a linen strip and graduated to feet and inches, while used in some rough work, has very little place in most surveying work, as it soon stretches or shrinks so as to make it too inaccurate for reliability.

The metallic tape is made by weaving fine bronze wires lengthwise through a heavy linen strip. This strip, usually fifty feet long, is painted a light color and the foot marks and fractional division are printed on it. The bronze wires are supposed to prevent undue shrinkage and stretching, but they serve their purpose indifferently. Such tapes, however, are found very useful

in taking short measurements on offsets, and for road cross-sectioning. These tapes may be obtained either mounted or unmounted. The reel is usually enclosed in a heavy leather box. If the tape is to be used where mud and water are encountered it should be taken off the reel as the box will soon becomed choked up with debris of various kinds carried in by the tape. The tape should be carefully dried before it is replaced on the reel.

The steel tape is the most useful of all the measuring devices to the surveyor. There are many varieties of steel tapes ranging from very light "ribbon" tapes to heavy tapes of the ribbon or "wire" type. The light tapes are thin strips of spring steel graduated to feet and inches or to feet and decimals (the common types to hundredths of a foot). Such tapes are never over 100 feet long and seldom over 50 feet.

Heavy steel tapes such are are used in most surveying are heavy strips of spring steel graduated to feet with the first and last foot divisions on the tape graduated, usually, to tenths of a foot, though they may be purchased with the end feet graduated to hundredths of a foot. The graduations are marked in different ways ranging from 0 feet to 100 feet.

In the heaviest patterns the tapes are graduated every five or ten feet with the last five or ten feet graduated to feet and the last foot subdivided to tenths of a foot. These tapes are often from 500 to 1000 feet long.

The light tapes are usually obtained on metal reels while

the heavier patterns are usually unmounted. It is possible to purchase reels for the heavy tapes. Such a reel should be of rather large diameter and of the "open" pattern.

The Gunter's chain (though fast becoming obsolete) is a sturdy measuring device 66 feet in length. It is made up of a series of links with metal handles at both ends. A link, so-called, is composed of a long bar with a small oval link at each end. The length of the three parts of the link taken together is 7.92 inches, or 1/100 of the length of the chain. The end links are still more complex, the bar link being divided into two parts. On the outer end of the last portion of this link is fastened the handle. The handle is so fastened to the link that it forms a part of the length of the link and is adjustable by means of a nut and screw, making it possible to correct the total length of the chain. Every tenth link is marked with a metal tag - at the tenth link is a tag with one point, at the twentieth link a tag with two points, etc. The fiftieth link is marked by a tag of some peculiar design, usually round. Since the chain is marked in a similar manner from both ends, care must be taken to determine whether or not the measurement is less or more than fifty links for fractions of chain-lengths.

The engineer's chain is similar in type to the Gunter's chain, the only important difference being that this chain is 100 feet long and the links are 1 foot in length. Both of these chains have been replaced by the more convenient and reliable steel tape

Among the many useful measuring devices used by the engineer or surveyor is the stadia. This is a combination of crosswires in the telescope of the transit or plane table alidade such that the intercept on a rod, as determined by these crosswires, multiplied by a certain constant will be the horizontal distance from the instrument to the point where the rod is held; It is not a perfect method, out under some conditions it will be found to be quite accurate as compared to measurements with the tape.

For approximate determination of distances, pacing, in conjunction with the pedometer is found to be very useful. The pedometer is a watchlike instrument which records, in one type, the number of paces taken by the person carrying it. In another type the record is the distance in feet, yards, metres or miles - in this pattern it is possible to regulate the instrument to the average length of the carrier's pace.

(20) MEASURING OF LINES

The measurement of lines with the ruler and with similar devices has but little place in the ordinary routine of the surveyor's work. On the other hand, the measurement of distances by pacing or with the tape and chain or by means of the stadia forms a large part of the many problems he meets. Of the last method more will be said in a later portion of the course.

(21) PACING

Pacing is much more useful than commonly imagined, a remarkable degree of accuracy being obtainable. Pacing serves a very useful purpose as a means of checking more accurate measurements

so that gross errors will not creep into long distances, e. g., such as dropping a tape-length, etc.

To pace a distance properly it is first necessary to determine the length of pace or the number of paces in an integral number of feet. Some surveyors prefer to develop an artifical pace of three feet, but in general it is probably better to maintain a customary length of pace and to determine the number of paces in 100 or 1000 feet. This is done by measuring a distance with a tape, say 1000 or 2000 feet, over ground representative of that over which the pacing is to be done and then walking the length of the line several times counting the number of paces. The average number of paces in 1000 feet will give a fair value to use in subsequent work.

If the ground is not too uneven or too steep the average distance obtained by pacing the lines in both directions is a fairly accurate measure of the horizontal distance.

(22) CHAINING OR TAPING

Over level or nearly level ground chaining or taping is carried out in the following manner:

Two men called rear and head chainman, respectively, are equipped with a tape, a set of eleven chaining pins, and a line rod. The head chainman should be the more experienced and reliable of the two men.

If a line rod or other signal is necessary, it is set immediately behind the mark at one end of the line in line with the two marks at the extremities of the course to be measured. This

line is, say, between 700 and 800 feet long.

The rear chainman takes his position to one side of the line at the end of the line away from the rod. The head chainman gives the rear chainman the zero end of the tape and unwinds the tape as he moves in the direction of the rod until the tape is fully extended. In his hand he carries the set of pins - these pins, eleven in number, are simple, round, wire darts with a ring bent on the upper end. They are used to mark temporary points on the ground during the process of chaining or taping.

When the head chainman has stretched the tape to its full length he takes one pin in his right hand, holds it against the tape at the 100-foot mark, and places himself to one side of the line facing the tape and holding the tape tout and horizontal.

The rear chainman then signals or directs the head chainman until the tape coincides with a line between the marks on the ground. When the head chainman is on line the rear chainman signals "down" or "stick" and the head chainman presses the pin into the ground in an inclined position at right angles to the line being measured. When this is done and the distance checked he answers the rear chainman by saying "down" or "stuck".

The rear chainman then drops his end of the tape and moves to the point just occupied by the head chainman and the head chainman moves forward to a new position on line dragging the tape behind him. The rear chainman watches the tape until the zero end is about five feet behind the first pin set by the head chainman. Then he calls "chain" or "tape" as a signal to the head chainman to stop.

This will bring the zero point very close to the pin. As the head chainman walks along the line he should "line" the rod in with some mark on the landscape and keep this mark covered by the rod so as to keep himself nearly on line.

The process carried out for the first 100 feet is repeated, the head chainman setting a second pin to indicate the end of the second 100 feet. When the head chainman in certain that the pin is properly set he calls "stuck", and the rear chainman pulls up the first pin, which he carries forward with him as he moves forward.

This procedure is continued until the head chainman, in dragging the tape forward, passes the rod set at the end of the line. This indicates that the last measurement is less than a full tape length. If the tape is graduated to tenths or hundredths of a foot throughout, the rear chainman places the zero end of the tape at the last pin set and the head chainman moves back along the tape until he is opposite the mark at the end of the line. The graduation on the tape immediately over the mark is observed, and to get the full length of the line, this length is added to 100 times the total number of pins placed in the ground by the head chainman. E. G. - if the line is 765.4 feet long there will have been 7 pins set in the ground, six of which are in the rear chainman's hand and one remaining in the ground, 65.4 feet being the fractional tape length last measured, hence:

$$(100 \times 7) + 65.4 = 765.4 \text{ feet.}$$

In most cases the tapes are constructed with the first foot and last foot divisions only, graduated to tenths of a foot, while the main portion of the tape is graduated to feet. When this condition exists the rear chainman holds the zero end of the tape at the last pin set until the head chainman determines the position of the nearest full foot mark beyond the mark at the end of the line. The head chainman then holds this foot mark at the end of the line while the rear chainman pulls the tape backward until it is stretched taut. The distance from the first foot mark to the pin is noted and gives the fraction of a foot to be added to the length indicated by the foot mark held by the head chainman minus one foot. To illustrate - if the distance from the last pin to the end of the line is 65.4 feet, the nearest foot mark beyond the end of the line is 66.0 feet. This point is then held on the end of the line and the tape pulled back until the 0.6 foot mark is opposite the pin near the rear chainman or 0.4 foot back from the 1.0 foot mark. Thus:

$$66.0 - 1.0 + 0.4 = 65.4 \text{ feet}$$

A new tape made by the Lufkin Rule Company is made 101 feet long, having two feet at the zero end of the tape graduated to tenths of a foot and the zero point placed at the middle point of these two feet. The tenths marked on the first foot run backward from the zero. This type makes the above arithmetic process unnecessary.

The process of measurement just explained is for tape measurements, though it is applied to measurements with a surveyor's or an engineer's chain. The sole difference is in the units employed

and the accuracy possible. With the chain measurements are seldom taken closer than to the nearest one-tenth of a link.

Should the line be longer than 1000 feet the work continues in the manner outlined until the head chainman has placed the eleventh pin in the ground. He then signals the rear chainman for pins and the rear chainman comes forward with ten pins in his hand. The rear chainman carefully counts the pins and the head chainman makes an independent count to check the tally of the rear chainman. If these counts agree the rear chainman, as recorder, marks 1000 feet in his fieldbook. The eleventh pin marks the position of the first hundred feet on the next thousand feet to be measured and will subsequently be counted in the tally of the second group of pins.

A method used by some surveyors is to start the rear chainman with one pin in his hand the the head chainman with ten pins. Thus, the pin in the rear chainman's hand records or tallies the first hundred feet measured, the second pin the second hundred feet measured, etc., until the last fractional tape length is determined. The rear chainman, in this method, counts only those pins he holds in his hand and does not include the last pin in the ground. It will be seen that in the measurement of lines over 1000 feet long the tally of pins at the 1000-foot point will be the same in either method used. Both methods are good, the one employed being a matter of choice on the part of the surveyor.

(23) HORIZONTAL MEASUREMENT ON SLOPING GROUND

This is much more difficult of accomplishment. In addition to the tape, chaining pins, and rod, each chainman should be supplied with a plumb-bob, the string on the same to be about seven feet long.

Let it be desired to measure the length of the line AB from A to B down the slope, Fig. 1.

The rear chainman takes his place at A as in the example below, while the head chainman moves down the slope until he stretches the tape out to its full length.

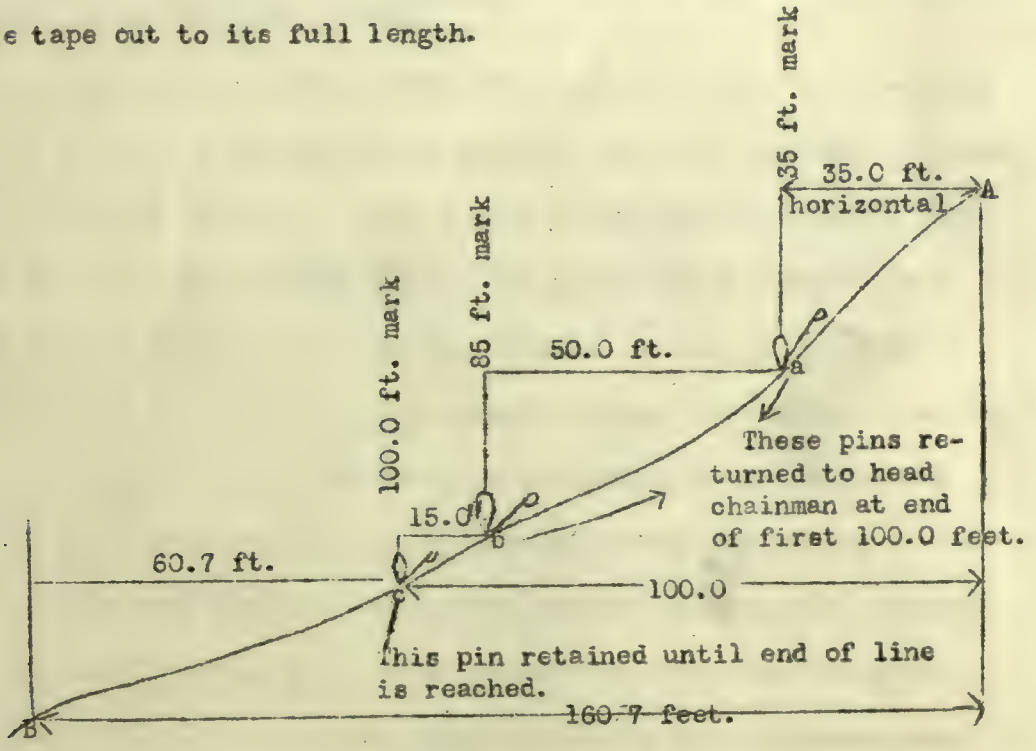


Fig. 1.

He then moves back up the slope to a point like a, a full number of feet from A, a being chosen at such a distance from A that it will be possible for the head chainman to hold the tape horizontal.

Say the distance along the tape is 35.0 feet. At this point the head chainman passes the string of the plumb-bob over the tape and allows the plumb-bob to drop until it is a short distance above the ground while the tape is held horizontal. The tape is brought to the horizontal by estimation, by comparison with horizontal lines in buildings nearby, or by use of the hand level. The rear chainman holds the zero end of the tape at A and lines the head chainman in. When the approximate position of the point is determined a space should be cleared of grass and debris and smoothed off so that a clear mark can be made.

The rear chainman checks alignment and, when the plumb-bob comes to rest, gives the usual signal. The head chainman releases the plumb-bob which will make a mark in the ground directly under the 35.0 foot mark on the tape. The plumb-bob is removed from the mark on the ground and a pin inserted in its place. (Where hard ground is encountered the plumb-bob must be carefully lowered to the point to be established and a scratch made with the pin which is then laid on the ground, point toward the mark.)

The rear chainman then moves forward to the point occupied by the head chainman and takes the foot-mark held by the head chainman, i. e. the 35.0 ft. mark, while the head chainman moves further down the slope to a new point b, say 50.0 feet away on line. The mark on the tape is noted to be 85.0 feet.

The rear chainman holds the 35.0 foot mark at the pin first set by the head chainman and the head chainman places a pin in the

ground under the 85.0 foot mark in a manner similar to that used in placing the first pin.

The rear chainman again moves forward with the first pin in his hand and takes the 85.0 foot mark from the head chainman. The head chainman moves down the slope to the end of the tape and a point is set under the 100 ft. mark on the tape. It is thus possible to sum up mechanically the several short lengths of tape used and to tally full hundreds, without much danger of arithmetical mistakes.

The rear chainman moves forward to the pin marking the 100.0 foot point on the ground where he hands the two intermediate pins to the head chainman who then moves forward along the line dragging the tape out until the zero end is at the first 100.0 foot pin.

This method of measurement is carried out until the line is measured. The last fractional part of a tape length, i. e, the portion from c to B in the figure, is determined as in the case of measurement of the last fractional tape length on level ground. The length is finally determined by counting the pins representing the full hundreds of feet and adding to 100 times this number the fractional length last obtained.

The measurement of horizontal distances up the slope can be made by having the rear chainman hold the plumb-bob over the points in succession and the head chainman move to points such that the tape can be held horizontal by the rear chainman. To measure downhill is a more accurate procedure than to measure uphill, owing to the greater difficulty in the latter case experienced

PROB. NO. I. MEASUREMENT OF DISTANCE WITH 100-FOOT STEEL TAPE

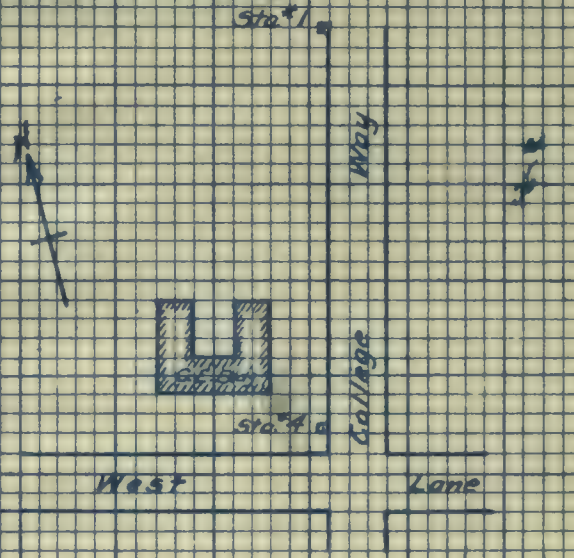
(1)

Course	Observed Length	Mean Length	Correction Ft.	Corrected Length
1-4	311.74			
		311.70	+0.09	311.79
4-1	311.66			

Equipment:
100-foot steel tape #19
Hand Level
Chaining pins
2 Plumb bobs
2 Line rods

August 20, 1907.
Clear and warm.
Jones, K.E.
Larkin, B.T.

Sta. #1 is a punch mark in center of cap on gas pipe set in concrete monument in College Way
Sta. #4 is a similar mark 10 feet from corner of College Way and West Lane.
Tape compared with University Standard and found to be 100.03 feet long.



by the rear chainman in trying to hold the plumb-bob over an established point on the ground.

Where uneven ground is encountered it is sometimes necessary for both chainmen to use plumb-bobs to make it possible to keep the tape straight and horizontal, e. g. - a stump may be on line or a small hummock may lie between the extremities of the tape.

(24) SLOPE MEASUREMENTS FOR DETERMINING HORIZONTAL DISTANCES

These measurements may be made in two ways. Both are rather laborious and expensive, involving the use of the level in one case and of the transit in the other.

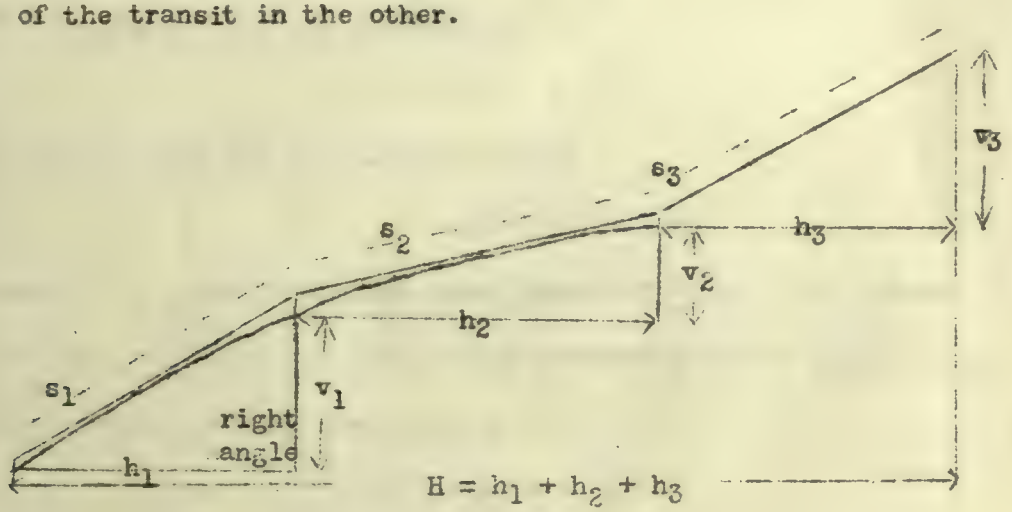


Fig. 2.

In the first method stakes are set along the line at intervals less than a tape length and the difference in elevation in feet is determined between successive stake tops by levelling. The distances on the slope between successive stake tops are then measured. From these measurements the hypotenuse and the vertical leg of a right triangle will be known.

By geometry there results - in general - see Fig. 2

$$h = \sqrt{s^2 - v^2} \quad \ldots\ldots\ldots (1)$$

h being the horizontal distance, s being the slope measurement and v the difference in elevation of the two points.

Solving for each partial horizontal distance and summing the quantities, the total horizontal length will be

$$H = h_1 + h_2 + h_3 + \ldots\ldots + h_n \ldots\ldots \quad (2)$$

In cases where the differences in elevation between successive points are small, an approximation to the right triangle formula will be found quite accurate enough and simpler to use. This formula is derived as follows:

From Eq. (1) may be written

$$v^2 = s^2 - h^2$$

the second term may be factored making

$$v^2 = (s + h)(s - h)$$

Since s and h are nearly the same length if v is small, assume that they are equal and apply this assumption to the first parenthesis, only, calling them both s.

Then $v^2 = 2s(s - h)$

Whence $s - h = \frac{v^2}{2s}$

and $h = s - \frac{v^2}{2s} \ldots\ldots\ldots \quad (3)$

The accuracy of this formula may be seen for the extreme case where s = 100.000 feet and v = 15.000 feet. The accurate solution gives h = 98.869 feet while the approximate formula gives h = 98.875 feet, this difference being well within the common limit of allowable error, e. g. - 1 part in 10,000.

To find the horizontal method by the second method stakes are set along the line in the same manner and the vertical angles measured with a transit. These vertical angles are measured from the horizontal line through one stake to the sloping line drawn from the same stake to the next stake on line, see Fig. 3. These angles are represented as α_1, α_2, α_3, α_4, etc. The slope distances, s_1, s_2, s_3, etc., are measured.

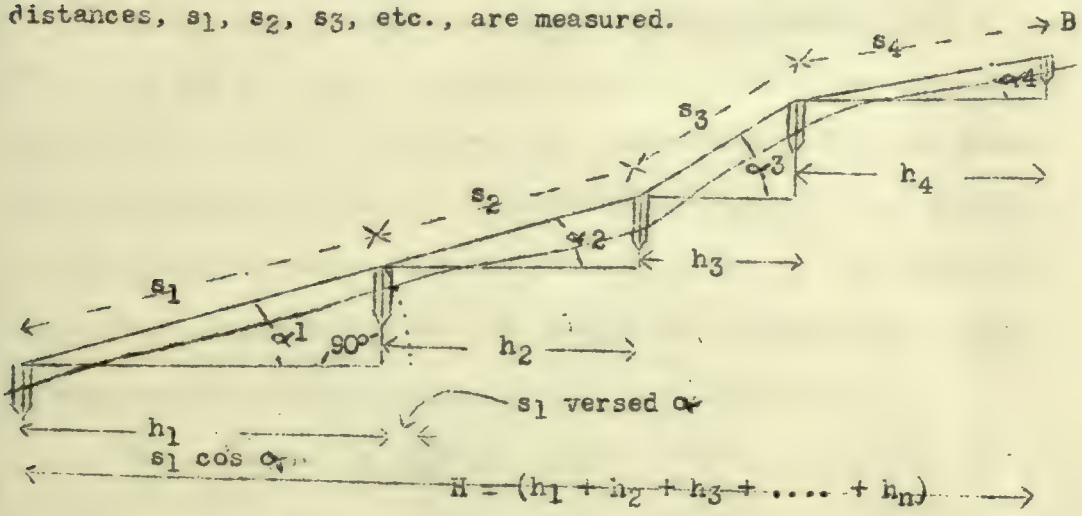


Fig. 3.

From the relation of parts in a right triangle it follows that $h_1 = s_1 \cos \alpha_1$ ..... (4)

and $H = h_1 + h_2 + h_3 + \ldots + h_n$ ..... (2) as above.

Where small angles are encountered and the slide rule is to be used it is often convenient to use the following formula:

$$h_1 = s_1 - s_1 \text{ vers } \alpha_1 \quad \ldots\ldots \quad (5)$$

The chief advantage in the use of this formula lies in the smaller number of significant figures necessary in obtaining the same accuracy as with the cosine. Wherever a table of natural cosines is available it is very simple to obtain the versed sine

by remembering the relation

versed sine = 1 - cosine.

This method is also good for solutions involving arithmetical computations with natural functions.

INSTRUCTIONS CONCERNING SOLUTIONS OF PROBLEMS AND THEIR ARRANGEMENT

At the end of each assignment a group of problems is placed which you are to solve. These problems will embody the principles set forth in this and the preceding assignments. When you solve these problems they must be returned as evidence of your mastery of the matter in hand. When the problem group has been examined and errors indicated, it will be graded and returned with copies of the correct solutions attached for your comparison.

The solutions of problems should be worked out upon the special paper furnished by the Associated Students' Store, Berkeley. All solutions are supposed to be your original work, not neat copies of work done on other paper. All numerical work and description must be shown in ink.

A problem should be solved in such a manner that each step will appear on the problem sheet (minor arithmetic processes and interpolations excepted). Tabulate a series of related partial or similar results.

Sketches or diagrams should be made to illustrate the problem whenever possible. These sketches may be made in pencil, but all lettering and dimensions must be shown in ink. In making sketches reasonable proportions should be observed and these should

approximate the conditions given; it is not necessary, however, to draw sketches or figures to scale, except when specially directed to do so.

Neatness and general arrangement of parts of a problem must be considered. You should lay out the method of procedure before starting to solve a problem and then work out each step in a logical order. Five minutes spent in "blocking out" the work will save much labor and worry. A surveyor should learn to be accurate and as direct as possible, since most of his work is of a nature requiring much time in execution and accuracy in results.

Problems will be graded on neatness, arrangement, completeness, and correctness - each part of the above list holds a weight of one-fourth of the total grade possible.

References:

Tracy, pp. 31 - 38.

Breed & Hosmer, pp. 11 - 18, vol. I.

Johnson, pp. 5 - 10.

Raymond, pp. 13 - 22.

PROBLEMS

1. A man paces a given line in both directions. The pedometer carried by him records 1355 paces in one direction and 1360 in the other. If his pace is 30 inches long, what is the length of the line (a) in feet? (b) in rods? (c) in miles? (d) in metres?

2. The horizontal distance between two points, A and B, is desired. Beginning at A the course was divided into the sections A-1, 1-2, 2-3, 3-B and the corresponding quantities s and v measured, in feet.

Section	s	v	$\frac{v^2}{2s}$	h	H
A-1	98.76	5.63			
1-2	99.00	10.70			
2-3	97.42	7.11			
3-B	67.10	3.24			

Compute the several horizontal distances, h, and the total length H, to the nearest one-hundredth of a foot, using approximate formula. Tabulate results.

3. To find the horizontal distance between two points, A and B, slope distances and corresponding vertical angles were measured.

Section	s		cos. method h	H	Vers. method h	H
A-1	73.65	5°10'				
1-2	98.65	10°56'				
2-3	100.05	26°40'				
3-B	85.00	31°00'				

(a) Solve for the several quantities, h, by the formula involving the cosine. Use 5-place logarithmic tables and logarithmic functions. Give results to nearest one-hundredth of a foot.

(b) Solve for the several quantities, h, by the versed sine formula, in each case do not use more than three significant figures in the versed sine, e. g. - 0.00376, 0.0986, or 0.173. Use natural functions and solve arithmetically.

(c) Tabulate the two sets of results and find H from each set.

UNIVERSITY OF CALIFORNIA EXTENSION DIVISION

Correspondence Courses

Surveying-1A Elements of Surveying Swafford

Assignment 3

ERRORS IN LINEAR MEASUREMENTS

FOREWORD:

In this assignment it is intended to give an elementary outline of the errors occurring in linear measurements, their sources, their correction, and their avoidance.

(25) THE TRUE LENGTH OF A LINE

The true length of a line can never be definitely known from measurements made by human agency. Mathematically it is possible to state that a line is a definite length, eg. - a line may be said to be 25 feet long, and from the purely mathematical consideration this line may be considered just 25 feet long, no more and no less, but should a line of unknown length be measured by any means available to the human race the length may be found to be, probably, 25 feet plus or minus, a length due to some error which in the last analysis will be indeterminable. The magnitude of the error depends upon several factors, some of which may be dealt with definitely, while others are, in the main, irremediable.

(26) ERRORS

Errors may arise from two general sources, namely, imperfection of instruments, and imperfection of the human functions.

Under the first head may be listed, the erroneous length of a chain or tape, change of length due to a rise or fall of temperature, irregular graduation of tape or other scale, non-adjustment

of instrument, and such injuries to equipment as the kinking of a tape or chain and the bending of spindles or compass needles.

The effect of human imperfection is shown in several ways. Most of the errors arising from this condition are termed personal errors. This general group may be again divided into what are called mistakes and accidental errors.

Mistakes are errors that occur in the mind of the observer, as, the wrong foot mark will be read from the tape; an angle of 52 degrees will be read as 57 degrees; in long lines a tape length will be lost or added; in attempting to repeat an angle five times, the instrument man may turn off the angle six times. When sighting a long distance on a sunny day, a person may regard the bright side of a rod as the full width. When this supposed width of the rod is bisected, the line of sight is actually to one side of the true point. Here we have a mistake, not an accidental error.

Accidental errors are really small mistakes - mistakes which do not come from a confusion of thought, but from the imperfection of human sight and touch, or from inaccuracy in estimation. These errors are exemplified by the small errors created in setting chaining pins or in estimating fractional divisions on a tape or circular scale.

(27) CUMULATIVE ERRORS

Cumulative errors occur from imperfection of equipment or from known natural causes. A tape may be too long or too short. In the first case the measured length of the line will be shorter than the true length by the product of the error in one tape length

multiplied by the number of times the tape length is contained in the total length of the line. Should the tape be too short, the reverse would be true.

The cumulative errors common to chaining or linear measurements are:

1. Incorrect length of tape or other measuring device.
2. Change of temperature, above or below the standard.
3. Incorrect alignment, i.e., both ends of tape not held in a horizontal line or in the vertical plane including the ends of the line measured.
4. Tape not stretched straight and taut but with the ends still on the line.
5. Sag of the tape due to wind or lack of proper tension.

(28) CONSTANT AND VARIABLE ERRORS

Cumulative errors may be divided into two sub-classes, constant and variable errors. There is but one constant error, that due to the erroneous length of tape. All of the other cumulative errors are generally variable in their nature. Temperature is constantly changing; the same errors in alignment are not likely to be repeated; the sag of the tape will vary according to the tension in the tape. It is possible that the sag and the temperature effects will be constant under some conditions and therefore cause constant errors. Likewise the pull applied at the ends of the tape due to a misunderstanding of what the proper tension should be, may be in error by a constant amount.

Accidental errors are in their nature variable.

(29) CORRECTION OF THE MEASURED LENGTH OF A LINE

The correction for the several kinds of cumulative errors be determined by considering the nature of each error, its law, and its magnitude. While under certain conditions accidental errors follow a precise law, it is impossible to correct them by mathematical means. The only way to eliminate them is to adopt such methods as will make the possibility of such errors as small as possible. It is evident that they can not be entirely avoided.

(30) CORRECTION FOR FALSE LENGTH OF TAPE

The false length of a tape may be due to several causes. In the process of manufacture the attempt is made to make the tape just 100 feet long when the temperature is 62 degrees Fahrenheit, the pull 12 pounds, and the tape fully supported on a flat surface. This ideal result is seldom if ever realized; hence the tapes purchased from the best makers are usually slightly in error.

After long, continued use the tape is liable to be kinked in several places. If the kinks have been straightened the tape will probably be too long because of the permanent stretching of the material. If the kinks have not been removed the tape will probably be too short.

A tape may be broken and subsequently patched. When broken a portion of the tape may be lost; or the tape may be lapped in repairing; or, if a splice be made, the two ends of the break may not be brought into proper contact. This source of error in the length of the tape is one that should be carefully considered in each individual case, for the greater portion of the tape may be

reasonably correct while the error at the patch will affect only such measurements as shall include this part. It should be noted here that an apparently perfect tape or one with kinks in it may have greater error in one part than in another.

Any one or all of these conditions may occur in one tape.

In a chain there is the possibility of the long links becoming kinked and thus making the chain too short. On the other hand, there is a possibility of the many oval links and the rings at the ends of the long links becoming stretched or of the surfaces of contact between them becoming so worn that the chain will be elongated. It will be remembered that a chain has over 600 wearing surfaces. The adjustable handles at the ends of the chain are used to correct the total length of the chain but it evidently does not follow that such a means will correct fractional measurements.

If a tape is shorter than standard the measured length of the line will be too great. If the tape is longer than standard the measured length of the line will be too small. By this is meant that a tape nominally 100.00 feet long and so assumed in making the measurement will, in the first instance, be contained a greater number of times in the total length of the line, and, in the second, a lesser number of times that would a tape of standard length. (It should be observed, that the measured length of a line is a ratio:- the length of tape to length of line.)

There are two general cases which arise in this connection, one where the error in the length of the tape is uniformly distributed

throughout the tape, and another where the error is located at a certain place in the tape.

In the first case a simple mathematical relation may be set up between the false and the correct values by considering what has been said before. This may be stated as follows:

$$l_f \; ; \; L_f = l_c \; : \; L_c \quad \ldots\ldots\ldots \quad (6)$$

or, put into words:

The false (nominal) length of tape (l_f) is to the measured length of the line (L_f) as the correct length of tape (l_c) is to the correct length of the line (L_c).

L_c is the correct or adjusted length of the line, L_f is the measured length of the line, l_c is the correct or standard length of the tape and l_f is the false (nominal) length of the tape.

From (6) may be written

$$L_c = \frac{L_f l_c}{l_f} \quad \ldots\ldots\ldots \quad (7)$$

For example, assume that a line has been measured with a tape nominally 100.00 feet long and that when the tape has been compared with a standard it is found to be actually 99.95 feet long, the error uniformly distributed. The total measured length of the line is 375.66 feet. Substituting in (7)

$$L_c = \frac{375.66 \times 99.95}{100.00} = 375.47 \text{ feet.}$$

In the case where the error is concentrated at one place in the tape the problem is carried out in the following manner:

First: Determine the magnitude of the error and the portion

of the tape in which the error exists, i. e., in the first ten feet, in the vicinity of the fifty-foot mark, etc. Call the error e.

Second: Measure the length of the line in the usual manner, using the nominal length of the tape. Note at the last measurement whether or not that portion of the tape is used that contains the error.

Third: Count up the number of full tape lengths, each one of which is known to contain the total error, and call this number N.

Fourth: If the last fractional measurement involves the error, add one unit to N making the number (N + 1). If the last fractional measurement does not include the error add nothing to N (this also applies if the last measurement is a full tape-length and follows from the third step above).

Fifth: From the foregoing can be written

$$L_c = L_f \pm e(N + 1) \quad \ldots\ldots\ldots \quad (8)$$

in the case where the error occurs in the last measurement.

Or $$L_c = L_f \pm eN \quad \ldots\ldots\ldots\ldots\ldots \quad (9)$$

when the last measurement does not contain the error.

To illustrate (8): Suppose that the measured length of the line is 375.66 feet and that the length of the tape is 99.95 feet by a comparison with the standard, though nominally 100.00 feet long. Let it further be assumed that an error of 0.05 foot occurs within the first 75 feet of the tape. Then,

$$L_c = 375.66 - 0.05\ (3 + 1) = 375.46$$

To show the effect when the error is not in the last measurement,

let it be assumed that the error of 0.05 foot appears in the last 20 feet, all other conditions being the same as above. Then.

$$L_c = 375.66 - 0.05\ (3) = 375.51 \text{ feet.}$$

The plus-or-minus signs in (8) and (9) indicate that the tape may be either too long or too short and the correction is to be applied accordingly.

(31) CORRECTION FOR CHANGE IN TEMPERATURE

The standard temperature for most steel tapes is set at 62 degrees Fahrenheit. Since temperatures vary from day to day and from one hour of the day to another, it is a common phenomenon that a tape changes its length accordingly. Most steel tapes expand or contract at the rate of 0.0000063 to 0.0000065 of their length for every change of one degree Fahrenheit (this quantity is a ratio and is called the coefficient of expansion; see any text on Physics). Assuming the temperature of all parts of the tape to be the same at any one time the change of the length of any portion of the tape will be proportional to the change in temperature and to the length of the tape involved.

If it is assumed that T_s is the standard temperature for the tape and that T is the temperature of the tape at the time of measurement of a distance, also that one nominal tape length is measured, i. e,, 100.00 feet,

$$l_c = l_f \pm 0.0000065\ l_f\ (T - T_s) \ldots (10)$$

The plus-or-minus sign indicates that the correction be added when the temperature is higher than standard and vice versa.

It will be noted that if the temperature is higher than standard the tape will be too long, hence it will be contained too few times in the length of a line measured under these conditions. In other words the line appears too short, hence the principle set forth in (10) holds for the measured length of the line and it may be stated

$$L_c = L_f \pm 0.0000065\ L_f\ (T - T_s) \ldots (11)$$

The same rule regarding signs holds as for (10).

In making this correction it must be recognized that the temperature will in all probability vary considerably during the work of measurement. The usual practice is to take the temperature for each tape length measured, to correct each tape length, and then to find the total distance, though in certain types of work the several temperatures are averaged and the average is used in the application of the formula to the full length of the line.

As an illustration of the method assume that the measured length of the line is 375.66 feet, and that the average temperature is 78 degrees Fahrenheit. Substituting in (11)

$L_c = 375.66 + (0.0000065 \times 375.66\ (78 - 62)) = 375.70$ feet.

It is evident that a minute error will be introduced by using the false length of line or the nominal length of the tape, but this error is so small that it may safely be neglected in most surveying work.

(32) CORRECTION FOR ERRORS IN ALIGNMENT

Errors in alignment should be avoided, but where such a thing as intentional deviation from the straight line is necessary, use should be made of (3), Assignment II, page 17. The distance, in this case, should be measured on the hypotenuse of the right triangle and the perpendicular distance from the line to the end of the tape should be the short leg of the triangle. This correction is obviously for only those measurements on each side of the mark where the end of the tape is off the line, and for no others.

(33) CORRECTION FOR TAPE NOT STRAIGHT AND TAUT

Error in this case may be prevented by proper care in seeing that the tape is straight and taut.

(34) CORRECTION FOR SAG OF TAPE

When a cord or strip of material of uniform cross section and of perfect flexibility is supported in a horizontal position by suspension from two points it assumes a curved form in a vertical plane. This curve is called a catenary; and it is possible to compute its form mathematically. But since the equation of the catenary is awkward to use in general work, an approximation is made by assuming that the curve is a parabola. This assumption, not so much in error as might be imagined, gives, on the whole, a very satisfactory solution of the problem.

Professor R. S. Woodward derives the final equation for the correction of length of a tape in the U. S. Coast and Geodetic Survey Report of 1892. This equation must be accepted as correct until you have had a thorough training in Calculus and Engineering Mechanics.

This final form for the difference in span for the tape fully supported and for the tape supported at the ends only, may be written

$$c_s = \frac{1}{24}\left(\frac{wl}{p}\right)^2 \quad \ldots\ldots\ldots \quad (12)$$

Equation (12) is the correction for a full tape length when supported at the two ends of the tape. c_s is the correction for the full span given in the same units as l; l is the total length of span in feet; w is the weight of the tape in pounds per linear foot; and p is the pull in pounds applied at the ends of the tape. This equation will also apply for any span of the tape less than full length, if the span is substituted for l.

If the tape is supported at several points we must consider the sag effect per span. In such case the tape is divided into, say, n spans of x feet per span. If $n = 1$,

$$c_s = \frac{1}{24}\left(\frac{wx}{p}\right)^2 \quad \ldots\ldots\ldots \quad (13)$$

Sag tends to shorten the span of the tape and causes the measured length of the line to be too long, i.e., the tape is contained too many times in the line measured. The correct length of the line is expressed by the formula

$$L_c = L_f - C_s \quad \ldots\ldots\ldots\ldots \quad (14)$$

C_s in (14) is the summation of the several quantities c_s found for the several tape lengths measured. If N tape lengths, including the fractional tape length at the end of the line, be taken, then

$$C_s = c_{s_1} + c_{s_2} + c_{s_3} + \cdots\cdots + c_{s_n} \quad \cdots \quad (15)$$

One should not fall into the error of assuming that the several quantities, c_s, can be multiplied by N except under special conditions; for an inspection of the formula will indicate that the correction in each case depends upon the cube of the span, the number of spans into which the tape is divided, and the pull on the tape, all of which may vary. The weight is usually constant for the tape used.

Let the length of the line measured be 375.66 feet, the pull applied at each tape length measured be 20 pounds, and the weight of the tape per linear foot be 0.010 pound. The assumption is that the conditions in each of the first three hundred-foot measurements are the same. Then

$$c_s = \frac{100.00}{24}\left(\frac{0.010 \times 100.00}{20}\right)^2 = 0.0104 \text{ foot.}$$

For the last 75.66 feet, however,

$$c_{s_N} = \frac{75.66}{24}\left(\frac{0.010 \times 75.66}{20}\right)^2 = 0.0045 \text{ foot.}$$

From the above,

$$c_s = 0.0104 + 0.0104 + 0.0104 + 0.0045 = 0.0357 \text{ foot.}$$

and

$$L_c = 375.66 - 0.036 = 375.62 \text{ feet (to nearest 0.01 foot).}$$

Again, the nominal length is used but the error is so slight as to be negligible.

The error caused by the lateral sag of the tape under wind pressure is so variable that it is impossible to correct it. The effect of wind is to shorten the tape and make the measured length of the line too long. The only thing to do when the wind is blowing sufficiently strong to cause a large error is to stop work until conditions will allow good work to be done.

QUESTIONS

1. In measuring a line what four sources of error are liable to occur?

2. Explain the difference in method of applying error in length of tape:
   (a) When the discrepancy is uniformly distributed?
   (b) When discrepancy is due to a defect in one part of the tape only.

3. Why are measured distances made by leveling the tape? What other means may be used to accomplish the same object?

4. Distinguish between an error and mistake in taking measurements. When may a mistake be called a blunder?

5. How may pacing be used as a check on blunders in measurement of a line?

PROB. NO. 2. SURVEY OF FIELD WITH STEEL TAPE

(2)

Course	Observed Length	Correction Ft.	Corrected Length
AB	336.83	-0.07	336.76
BD	438.70	-0.09	438.61
DE	616.65	-0.12	616.53
EA	241.89	-0.05	241.84
BE	425.93	-0.09	425.84

Equipment:
100-foot steel tape #30
Hand level
Chaining pins
2 Plumb-bobs
2 Line rods

August 27, 1909
Cloudy and cool

Homer, W. L.
Morse, J. T.

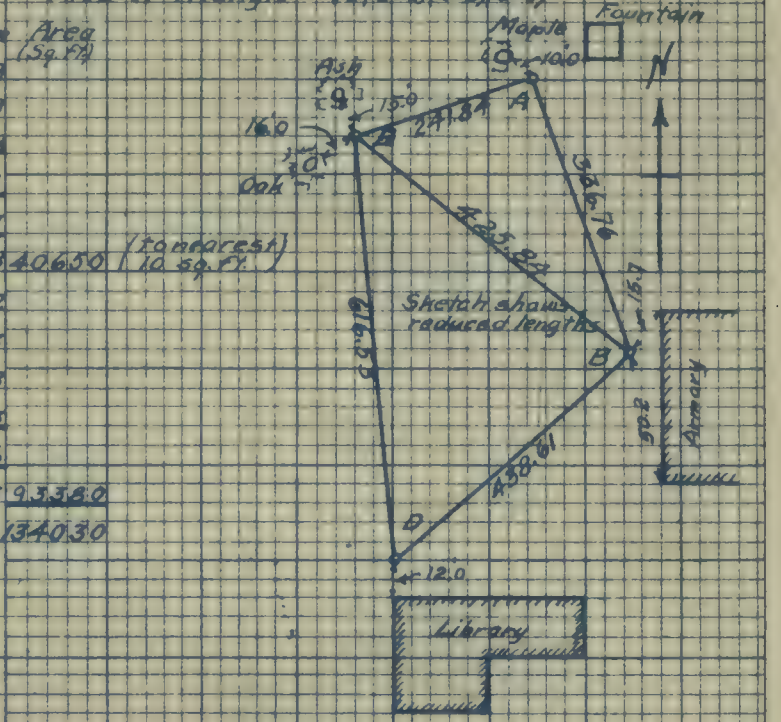
Divided field into two triangles and measured each of the five lines once.

Standardized tape — Length = 99.98 Ft.

Area of triangle = $\sqrt{s(s-a)(s-b)(s-c)}$

Computation of Area

Triangle	Line	Length	$s = \frac{1}{2}(a+b+c)$	s-a	s-b	s-c	Logarithm
ABE	AB=a	336.76	502.22				2.70089
	BE=b	425.84		165.46			2.21869
	EA=c	241.84			76.38		1.88298
	2	1004.44				260.38	2.41561
	s=	502.22				2	9.21817
							4.60908
						Area (Sq. Ft.)	40650 (to nearest 10 sq. ft.)
BDE	BD=a	438.61	740.49				2.86952
	DE=b	616.53		301.88			2.47983
	BE=c	425.84			123.96		2.09328
	2	1480.98				314.65	2.49783
	s=	740.49				2	9.94046
							4.97023
							93380
						Total Area	134030



UNIVERSITY OF CALIFORNIA EXTENSION DIVISION
CORRESPONDENCE-STUDY COURSES IN TECHNICAL SUBJECTS

Course 1/. Elements of Surveying Swafford

Assignment 4

ERRORS IN LINEAR MEASUREMENTS
(continued)

FOREWORD:

This assignment is a continuation of the discussion of errors in linear measurements with particular emphasis on accidental errors and the law governing them. A later portion of the assignment is devoted to a discussion of precision, discrepancies, and allowable errors.

(35) ACCIDENTAL ERRORS

Accidental errors have a tendency to balance each other; they are often called compensating errors. It is unfortunate that this term has been used, for it tends to mislead a person into believing that by some magic or other the effect of accidental errors is eliminated in all cases. In the greater part of the work done in surveying each quantity is measured but once, and furthermore, most quantities measured are relatively small. Where the relatively small quantity is measured many times or where large measurements are made so that there will be many partial measurements every accidental error will have a tendency to balance some other.

A mathematical study of the law of errors, particularly accidental errors, and the probability of errors (see Merriman's "Method of Least Squares") shows that such errors will come nearer and nearer to balancing each other as the number of measurements

approaches infinity. This means not that in the measurement of a single tape length, for instance, the accidental error will be eliminated but that the total length of a line measured by repeated trials approaches more nearly the true, unknowable length, the greater the number of times the line is measured.

It must not be supposed that, if a long line is measured by a great number of partial measurements, the total length will be more accurately determined than the length of a shorter line of fewer partial measurements made with the same care and under the same conditions. On the contrary, the actual error will in all probability be less in the shorter line.

The common accidental errors occuring in linear measurements are caused by

1. Variation of pull or tension in the tape.
2. Error in setting pins or other markers.

Mistakes or blunders may occur through

1. Incorrect count of tape or chain lengths.
2. Mistakes in reading the graduations.
3. Disturbing of pins or other markers after they have been set.

(36) CORRECTION OF ACCIDENTAL ERRORS

Of all the accidental errors listed the only one which may be corrected mathematically is that due to variation of pull or tension. This error lies on the border line between a true cumulative error and an accidental error, due to the fact that in most surveying the pull is estimated and not exactly measured.

The case does arise, however, where the pull is measured with a dynamometer or spring balance. When this is the case the

effect of pull comes into the group of cumulative and correctable errors.

It has been found that if a bar of steel one square inch in cross section and of any length l, is stressed by applying loads at the ends of the bar, the bar will be either stretched or shortened by $\frac{1}{30,000,000}$ of its length for every pound of load applied. Hence if the area of cross section of the tape and the magnitude of the pull applied are known

$$c_p = \frac{pl}{AE} \quad \ldots\ldots\ldots\ldots \quad (16)$$

This is an expression of Hooke's Law (see Church's "Mechanics of Engineering") in which p is the pull applied in pounds, l the length of the span of the tape used in feet, A the area of cross section of the tape in square inches, and E the modulus of elasticity (30,000,000 pounds per square inch). c_p will then be given in fractions of a foot for all cases that arise in surveying.

If in measuring a line the pull is determined for each tape length or fraction thereof, the corrected length will be found from

$$L_c = L_r + C_p \quad \ldots\ldots\ldots\ldots \quad (17)$$

C_p is the summation of all the quantities c_p, the same principle applying as in the correction for sag. It should be noted that while it is usually necessary to compute the pull correction for each separate tape length measured, there are many cases in which it is proper and better to correct the whole length of the line at one time.

To illustrate this let it be assumed that the length of line measured is 375.86 feet, pull on the tape 20 pounds, cross

section of the tape 0.0029 square inch, E = 30,000,000 pounds per square inch, and that the nominal length of the tape is 100.00 feet. It is further assumed that the same pull is exerted for each tape length or fraction. Then

$$C_p = \frac{20 \times 375.66}{0.0029 \times 30,000,000} = 0.086 \text{ or } 0.09 \text{ feet.}$$

and

$$L_c = 375.66 + 0.09 = 375.75 \text{ feet.}$$

Pull always lengthens a tape and makes the correction additive. A common case that arises is where a tape is found to be just 100.00 feet long under a certain tension. If the pull be below that assumed as standard, the correction will be negative for the difference in length found by computing the correction for each tension. This is but a step from the above method.

(37) NORMAL TENSION

If the two corrections, sag and pull, are made equal, it is possible to simplify the later computations by obtaining what is called <u>normal tension.</u> Hence placing

$$\frac{pl}{AE} = \frac{w^2 l^3}{24p}$$

the normal tension, $p = l\sqrt{\frac{w^2 AE}{24}}$ ..... (18)

If the value of p is found for a temperature of 62 degrees Fahrenheit so as to make the tape span just what the true length of the tape is when fully supported and unstressed, the saving in work will be considerable. A table or diagram may be constructed for a particular tape such that the normal tension for any span of the tape may be quickly obtained.

The normal tension above must not be confounded with the so-called normal tension which will make the span of the tape just 100 feet when supported at the two ends, for in the latter case the pull and sag will probably not balance each other, because the tape may not be just 100 feet long when unstressed and fully supported.

(38) PRECISION

While all surveying should be conducted with due care, all surveys need not be made with the same precision, and precision in some parts of a given survey should be more scrupulously sought than in other parts. The purpose of the survey, the time or its equivalent, the money expended in doing the work, will and should in a measure determine the refinement of methods and the character of instruments employed. For example, a compass and chain survey suffices for the ordinary uses of farm or timber lands, or for estimating crops, areas ravaged by fire etc., while the measuring out of a city lot would call for a transit and tape survey possibly to the last refinement. In the determination of a Base Line (see Johnson, pp. 447 et seq.) not only is every step made with extreme care, but every known source of error in measurement is considered; corrections are applied for alignment, sag, pull, temperature, and difference in level; and finally a reduction to sea-level is made, and many checks and balances are sought.

In railroad surveys the different degrees in precision may be aptly illustrated by the methods employed in the various steps, as for reconnaissance, preliminary, and location, while also in

the last, location, different degrees are further observed for separate parts of the work.

In cases where two or more determinations of the same measured quantity are made, any large discrepancy should be rejected and a mean of the fairer values taken. This does not imply that a mean of two or more observed values is correct, but that the possible error, too large in one case, is balanced by a possible error, too small in another. While this is not necessarily true, for both may be too large or both too small, it is regarded as a safe course to pursue where each value is determined with equal care. Furthermore, discrepancies serve to indicate mistakes, the most common source of wide discrepancies.

Ordinarily a tape that is known to be standard may be relied upon for most work. The distances measured on slope and reduced to level may also be relied upon when the inclination is exactly known. The character of the territory traversed - hilly, wooded, or marshy - calls for varying degrees of precision; it is not well to apply methods of refinement inconsistent with the nature of the land. If it is sufficient that the survey be correct in the ratio of 1 in 200, it would be folly to try for correctness in the ratio of 1 in 5,000, or 10,000 or more. Consistency is the essential element always.

No error known to exist should be permitted where it may be removed. If a traverse, for example, does not close by an amount that seriously affects the result, some lines, or angles, must be remeasured to determine the discrepancy.

Ideal conditions, such as measurement over open territory on firm and level ground are seldom realized. When such conditions do not obtain the surveyor must exercise great pains. A disregard of reasonable means to secure fair results is inexcusable. Over wooded areas many obstructions such as trees, brush, etc. are present, which must be removed unless corrections are applied to each tape length or line segment. Here the knowledge we may have of the nature of the obstruction must govern the result, or the judgment of the chainmen must be called upon.

It is often necessary to pass some such obstruction as a tree standing upon the line; in such case the correction $c = \frac{h^2}{2L}$ is necessary. This correction must be subtracted. Here h is the offset from the line at the point where it occurs and L is the length from the point on line to the offset.

On slopes the tape should be carefully leveled, preferably with a tape-level or by aid of a hand-level (to be described later on in the assignment on Leveling) and the end points of the tape should be transferred to the ground by means of plumb-bob. Where great precision is not required, the tape may be leveled with the eye, lining it in by the horizon or the lines of a building. A rude substitute for the plumb-bob is a chaining pin dropped from the point to the ground. Caution must be used in chaining on slopes, as these often appear greater than they really are. In ascending a slope it is more likely to misjudge than in descending. In "breaking" tape do not make the blunder of mistaking the wrong division mark.

The errors common to chaining may now be summarized. These are:

Error in Tape - longer or shorter than standard. If longer, the measured line is too short; add a correction. If shorter, the measured line is too long; subtract a correction.

Error, tape not horizontal - the correction $\frac{h^2}{2L}$ must be subtracted.

Error due to temperature - if temperature is above standard, tape is too long, correction must be added; if below standard, correction subtracted.

Error due to tension - if pull is below normal, line too short; add correction; if pull is above normal, subtract correction.

Error due to sag is a cumulative error that must be corrected by adding a calculated quantity.

Error in alignment is synonymous with that for tape not horizontal.

Do not make blunders, such as reading the wrong tape division, plumbing down from the tape ends, making wrong count of tape-lengths, or disturbing pins when once set.

References:

Tracy, pp. 39 - 46.

Breed & Hosmer, pp. 11 - 13, vol. I.

pp. 24 - 33, vol. II.

Johnson, p. 449 et seq.

Raymond, pp. 23 - 30.

UNIVERSITY OF CALIFORNIA EXTENSION DIVISION
CORRESPONDENCE-STUDY COURSES IN TECHNICAL SUBJECTS

Course 1A Elements of Surveying Swafford

Assignment 5

THE COMPASS AND ITS USES.

FOREWORD:

This following assignment outlines the terms used in compass surveying and gives a description of the compass and its use.

(39) THE MAGNETIC NEEDLE

The peculiar properties of the magnetic needle are known in a general way by nearly everyone. This needle is a slender bar of iron which has been magnetized and suspended upon a pivot so that it may swing freely in a horizontal plane and, within a limited amount, vertically.

If the needle is allowed to swing freely it will soon come to rest, pointing in a northerly direction. Should it be disturbed and again allowed to come to rest it will be found to point in the same direction as it did in the first instance. This is due to the fact that the earth is a great magnet with a positive or north pole in the region of the geographical north pole and a negative or south pole in the region of the geographical south pole.

The end of the needle which points to the north magnetic pole of the earth is the negative end of the needle. This end is called the North end of the needle because it always points toward the north magnetic pole. This north end, in the surveyor's compass can be easily distinguished, for in the north latitudes the south end of the needle has either a small brass sleeve or

a wrapping of fine brass wire fastened about it between the pivot and the extremity.

(40) THE DIP OF THE MAGNETIC NEEDLE

The wrapping of wire is necessary on account of the dip of the magnetic needle. When magnetized the north end swings toward the north magnetic pole and in doing so is drawn downward so that the needle assumes a final position in which the north end is much lower than the south end. This dip is zero at the Equator and increases until the needle stands in a vertical position at a point just north of Hudson's Bay. In a similar manner the south end of the needle will dip for positions south of the Equator. The wrapping of brass wire is used as a counterweight with which to balance the needle.

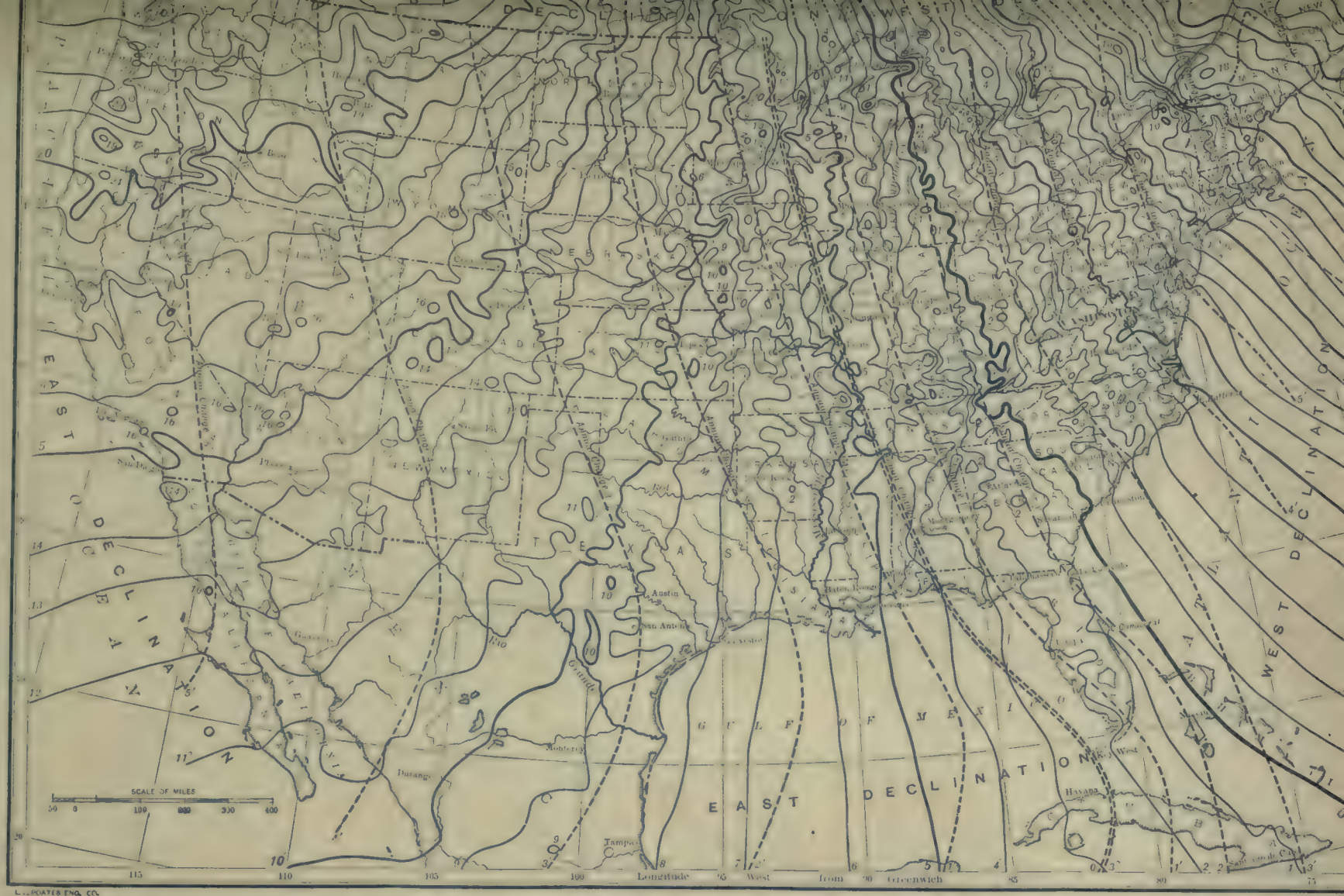
(41) THE MAGNETIC POLES OF THE EARTH

The magnetic poles do not coincide with the geographical poles. The geographic poles are the two points on the earth where the axis of rotation pierces the surface. The north magnetic pole lies just a short distance north of Hudson's Bay, Canada, while the south magnetic pole is just south of Australia. In general, then, the magnetic needle does not point toward the true or geographic north pole. Since the term north is used rather loosely by everyone, it will be well to give a clear definition of the terms used in this discussion.

True or geographic north or south will be indicated by the words "true north" or "true south".

Magnetic north will be called "magnetic north".

North, unqualified, will indicate the north direction as



L. POATES ENG. CO.

NOTE.— The lines of equal magnetic declination, or isogonic lines, are given for every degree, and are based on all known observations to date. They apply to January 1, 1910. Along the line marked zero, the magnetic needle points true north and south.

The north end of the compass is moving to the westward for places east of the line of no change, and to the eastward for places west of the said line, at an annual rate indicated by the lines of annual change (dotted lines). In other words, east of the line of no change, the isogonic lines are m westward; whereas, to the west of the line of no change, they are moving eastward.

applied in discussions involving either true or magnetic north, the limitation being set by the nature of the problem and method used.

(42) DECLINATION OF THE NEEDLE

Magnetic surveys of the earth's surface show that the pointing of the needle does not follow a regular law. The angle by which the needle points away from the geographic or true north is called by surveyors the declination of the needle, which is the same as the nautical term Variation. If the needle points to the east of true north, the declination is east and if to the west of the true meridian, the declination is west.

The declination of the needle is different for different localities on the surface of the earth. If an observer traversed a line along which the declination of the needle is constant, his path would be an irregular line. This line is called an isogonic line, a line of constant variation of declination of the magnetic needle. Figure 4 is a chart showing the positions of such lines for the United States.

There is a line extending from the region of Lake Superior to South Carolina which is marked 0°. This is the Agonic Line or line of no declination. If the needle is read at any point along this line the magnetic north will coincide with the true north. To the east of this line the declination is west and to the west of this line the declination is east.

(43) CHANGES IN DECLINATION

The declination of the needle is not constant. In fact, the declination of the needle is changing continually.

Secular variation is a slow change caused, apparently, by the slow shifting of the magnetic poles of the earth. The law of the change or its cause is not clearly understood. In the United States the change in declination from this cause is about three minutes per year on the average. This change is not the same for all localities; the greatest rate of change is near the Agonic Line and the rate diminishes as the distance from the Agonic Line increases. Lines indicating the annual change are shown on the chart.

Annual variation is a small change of approximately one minute of arc.

Diurnal or Daily variation is a change in the declination of the needle occurring each day. The amount of the variation depends upon the season of the year and the time of day, ranging from 3 to 12 minutes. The needle is in its most easterly position at about 8 a.m. and in its most westerly position at about 1:30 p.m.; the mean positions occur at about 10 a.m. and 5:30 p.m.

Irregular variations come from a variety of causes such as electrical or magnetic storms, during which the movements of the needle are so erratic that it is impossible to read it accurately and from local attraction, which will cause a definite deflection of the needle.

Of the variations in declination the most important are the Secular and Diurnal Variations. The Annual Variation is so small that it is unimportant. In the case of Irregular Variation due to magnetic storms, etc., all that can be done is to wait until the disturbance subsides. Local attraction is easily corrected in most cases.

(44) THE SURVEYOR'S COMPASS

The surveyor's compass is a device for obtaining the direction of lines. A compass box is mounted on a vertical spindle. The spindle turns in a hollow vertical sleeve at the lower end of which is a ball and socket joint. The whole is mounted upon a tripod so that the instrument may be set firmly over a point.

The tripod is provided with a loop or hook immediately under the center of the vertical spindle so that a plumb-bob may be suspended from it while setting the instrument up over a point.

The bottom of the compass box is a dial. In this dial, or on the frame carrying the dial, are two small spirit levels which are set parallel to the bottom of the compass box and therefore perpendicular to the vertical axis of the instrument; one level is parallel, the other perpendicular, to the line of sight. These are used in leveling the dial. The dial is divided into quadrants, one end of the line being marked N or with some conventional device to indicate north. Diametrically opposite is placed S for south. The ends of the line at right angles to the first line are marked E and W.

On a circle which is raised a short distance above the dial are found the graduations, usually degrees and half degrees.

At the point corresponding to north and to south the graduations are marked 0°, while the points E and W are marked 90°. The intervening divisions are conveniently marked in tens so that readings may be easily made.

On the frame carrying the compass box, or on the sides of the compass box, are mounted two standards or sights. These are vertical bars several inches high in which narrow slits are cut for sighting. In certain types of compass there are just two narrow slits, while in others there is a very narrow slit in one standard and a wider slit in the other standard in which is mounted a fine wire or thin metal strip. These standards are fastened to the compass box on the line marked N and S, which line is the line of sight of the instrument.

In the center of the compass box is a fine, needle-like pivot on which the magnetic needle swings. This pivot is very carefully centered and should be handled delicately. On this pivot swings the needle, supported on a jewelled bearing similar to the bearing on the balance wheel in a watch. The jewel, either of glass or of some semi-precious stone, is mounted at the center of the needle in a circular setting. It has a small conical depression cut in it into which the pivot fits.

The whole needle mechanism is very delicate; hence a lever is placed in the lower portion of the compass box by means of which it is possible to raise the needle off the pivot while the instrument is being moved from one place to another.

The compass box is covered with a thin sheet of glass to protect the delicate parts within from dust and damage.

(45) BEARING OF A LINE

In surveying with a compass the angular quantities determined by means of the compass itself are called bearings. The bearing of a line may be defined as the angle which the line makes with a line of standard direction called a meridian (i.e. either magnetic meridian or geographical meridian).

Bearings, either true or magnetic, are reckoned from the north to the east or west, and from the south to the east or west. As stated above, the graduated circle of the compass is marked from 0° at N and S to 90° at E and W, hence bearings range from zero degrees to ninety degrees only. To illustrate:

N 0° E (or W); N 15° W; S 89° E; S 11° W.

Where the bearing is exactly N or S or E or W, it may be so indicated by using for brevity the single letter, but it often prevents confusion to indicate such a bearing as in the first example above. Bearings are never more than 90° from the N or S, respectively.

The bearings are determined by the position of the needle with respect to the markings on the dial, and by the position of the line sighted. Usually magnetic bearings are obtained. These may be subsequently changed to true bearings if the magnetic declination is known for the time and locality (see Computation of Angles and Bearings, Art. 54, Assignment VI). A mechanical device known as a declination arc is found on some compasses, by

means of which it is possible to set the compass circle so as to obtain the true bearing directly. (See Declination Arc, Art. 48.)

(46) TO SET UP A COMPASS

To set up a compass over a point, the procedure is as follows:

1. The instrument man stands the compass before him so that the two legs of the tripod are toward him and the third leg away from him.

2. He grasps the two nearer legs in one hand and the third leg in the other, swings the third leg away from him till the angle between the legs is about 30°, places the foot of the leg on the ground about three feet beyond the point over which the instrument is to be set, and plants it firmly.

3. He then takes the other legs, one in each hand, and swings the whole instrument and tripod toward him, opening the legs of the tripod as he does so, until the center of the tripod is approximately over the point on the ground and the legs about symmetrically placed. The legs are then set on the ground.

4. The plumb-bob is then suspended from the loop on the tripod. In doing this care is taken to tie a "running bow knot" in the string sufficiently far below the loop so that the height of the plumb-bob above the ground may be regulated.

5. He determines the position of the plumb-bob with respect to the point on the ground and moves the feet of the tripod about until the plumb-bob hangs directly over the point. This shifting requires considerable practice before the surveyor becomes adept at it.

6. To level the instrument he grasps the compass box or frame (never the standards) firmly in both hands and tilts the head of the instrument about the ball and socket joint until both small bubbles in the level tubes remain in the center. As a cautionary measure it will be well to bring one bubble to the center of its tube at a time. He swings the head about the vertical axis until the bubble tubes are turned 180° from their former position and notes whether the bubbles remain in the center. If they do not he brings each bubble halfway back toward the center of its tube, making the dial of the compass level and the vertical axis truly vertical. The reason for this will be apparent after the Adjustment of the Level Tubes has been read.

7. He releases the needle by means of the lever, and allows it to swing freely. The instrument is then ready for use.

(47) TO TAKE THE BEARING OF A LINE

The instrument being set up over one end of a line, it is possible to obtain the bearing of the line as follows:

1. The needle, for any locality, points in a fixed direction, namely, toward the magnetic pole. (Magnetic north.)

2. The line of sight, as determined by the slits in the standards, swings about the vertical axis past the needle.

3. The dial with graduated circle, rigidly fastened to the standards, swings about the vertical axis under the needle.

It must be kept clearly in mind that the needle is stationary for any one "set-up" and that the dial and the line of sight move.

In placing the E and W on the dial their positions are reversed from the east and west points of the horizon. This follows directly from the three considerations immediately above and will be clear after a reference to Figures 5 and 6.

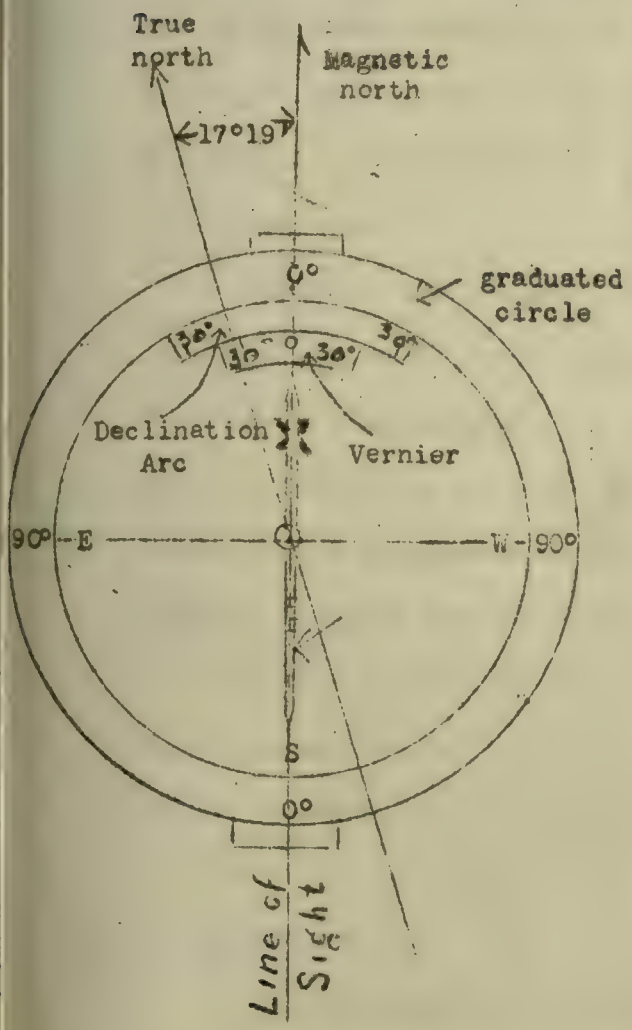


Fig. 5

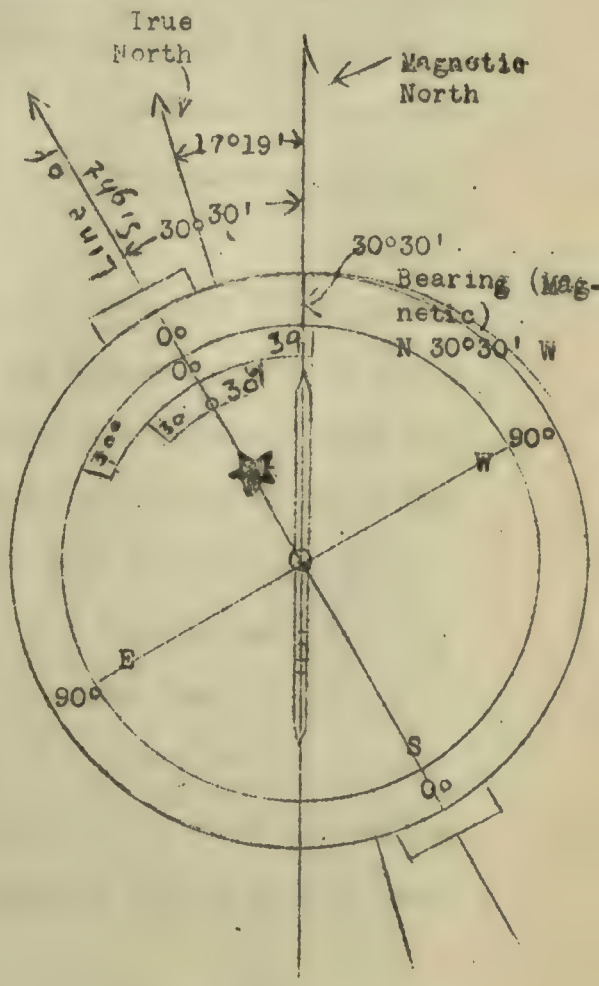


Fig. 6

Figure 5 shows the compass box set so that the line of sight points toward the magnetic north, in which case it coincides with direction of the needle. The magnetic declination is taken as

17° 19' East and is so shown on the figure, i. e;, magnetic north is east of the true north.

To obtain the bearing of a line, sight on a signal at the far end of the line from where the instrument is set up. Sighting consists of turning the compass box until the slits in the standards are in the same vertical plane with the line on the ground, that is, ths signal will appear in the middle of the farthest slit or will be covered by the metal strip in that slit when sighting through the nearer slit.

Assuming that Figure 6 represents these conditions, it is seen that the needle still points to the magnetic north and that the line of sight points in a northwesterly direction. Since the zero has moved with the line of sight the North end of the needle points toward a graduation, say 30° 30' from the zero but to the right or eastward from 0°. The line has a bearing that is really N 30° 30' W. An examination of the dial shows that the 90° point in the right-hand portion of the dial is marked W, as above indicated. It is now apparent why the letters E and W are reversed on the dial. The bearing shown is the magnetic bearing of the line.

(48) THE DECLINATION ARC

The declination arc is a device which makes it possible to take the true bearing of a line directly. This arc is fastened rigidly to the portion of the compass box which carries the full, graduated circle and extends about 35 degrees in both directions from zero. It is commonly graduated to half degrees. This scale is similar to the main scale of the compass.

Fastened to the portion of the compass box carrying the dial is a small vernier scale, or merely an index line. The vernier scale is graduated from zero, in the middle, to thirty in both directions. (See Theory of the Vernier.) By means of the index mark or vernier it is possible to set off an angle on the declination arc.

In Figure 5 the declination arc is shown in the "North" position of the compass box. The declination arc is so set on the compass box that when the zero on it coincides with the zero of the vernier or the index mark, the zero on the main circle lies in the line of sight. This is true no matter where the declination arc may be fastened to the compass box.

The line of sight is connected with the portion of the box carrying the dial and vernier or index mark.

Referring to Figure 7 let it be desired that bearings taken with the magnetic needle be the true bearings of the lines. The needle will always point toward the magnetic north. From the figure it will be seen that if the needle is to read zero degrees when the line of sight is pointing toward the true north it will be necessary to swing the line of sight WEST of the zero on the graduated circle. This is accomplished by the use of the declination arc and vernier. Using the same magnetic declination as before the setting is shown in the figure. The needle points to zero and the line of sight points to the true north. It must be noted that the marks indicating the cardinal points of the compass

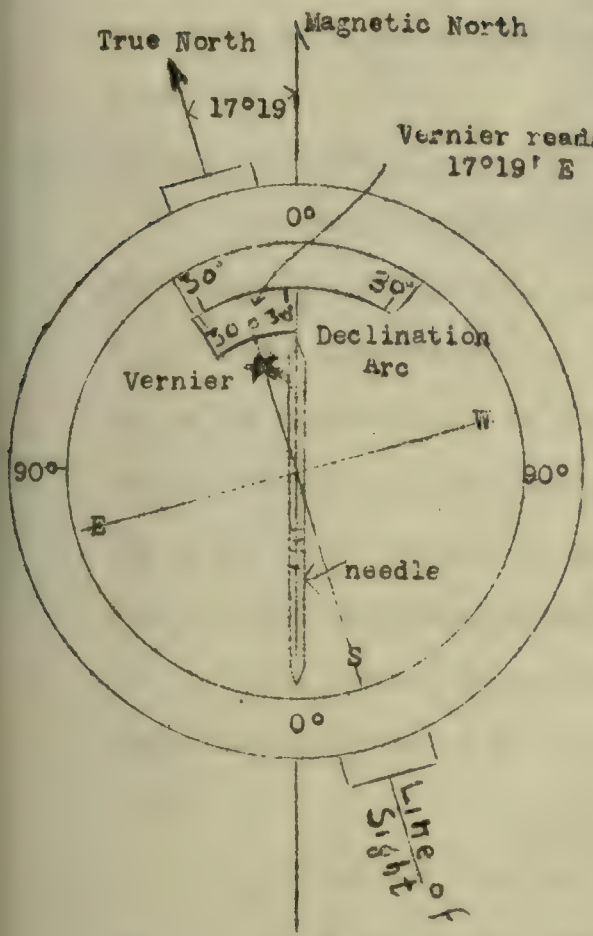


Fig. 7

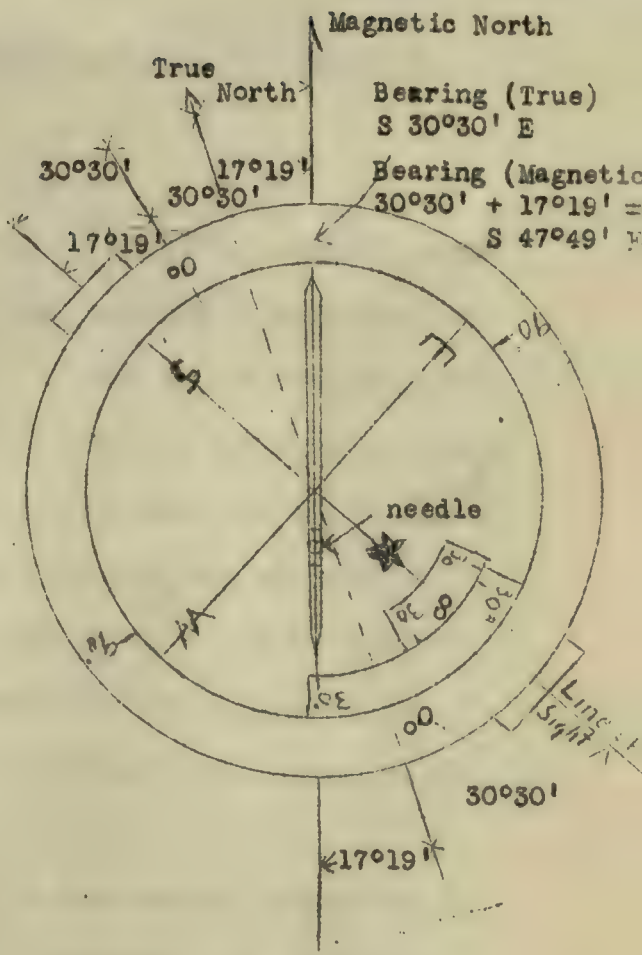


Fig. 8

turn around under the graduated circle with the line of sight so that readings should be referred to the graduations on the circle. The markings on the dial are used to indicate the general position of the line of sight, only.

This setting is made but once for any locality and the parts should be clamped securely so that it will not be changed during the course of the survey in all cases in which the declination arc is set off as explained above.

Figure 8 shows the compass set for a bearing of S 30° 30' E. The relative positions of the parts of the compass should be

carefully noted and compared with their positions in Figures 5, 6 and 7.

(49) READING THE COMPASS NEEDLE

This is a simple operation, but it should be carefully done. Most compasses are now made so that the sighting is done from the S end of the dial through the N end. If this be the case the north end of the needle should be read at all times to determine the bearings of the lines sighted. That is, always place the eye in sighting at the south standard in whatever direction the sighting is made and read the graduated scale at the north end of the needle.

If the compass be so constructed that it is possible to sight from either the S or N end the following rules should be observed:

1. If sighting from the S end of the compass through the N end or if the N end of the compass is toward the point sighted, read the north end of the needle.

2. If sighting from the N end of the compass through the S end or if the S end of the compass is toward the point sighted, read the south end of the needle.

In reading the bearing it is always best to look along the needle from the end opposite from the one being read. This prolongs the line of the needle until it intercepts the graduated circle and eliminates parallax or apparent sidewise displacement of the needle.

- -

References:

Tracy, pp. 293 - 300.
Breed & Hosmer, pp. 16 - 30, vol. I.
Johnson, pp. 13 - 37.

PROBLEMS

1. The magnetic bearing of a line is S 15° 30' W. The declination of the needle is 11° 15' E. Draw a figure similar to those shown in the assignment to show the position of the compass with respect to the line and with respect to the true meridian. Make the dial three inches in diameter, outside dimensions.

2. Draw a diagram showing the compass set to give the true bearing of the line shown in Problem 1. What is this bearing?

3. Draw a diagram showing the compass sighted along a line, the true bearing of which is N 56° W, so as to give the magnetic bearing. If the declination is 10° W, what is this bearing?

4. The declination of the needle at the present time at a certain place is 17' W. The secular variation is 5' eastward per year. What will the declination of the needle be in five years?

UNIVERSITY OF CALIFORNIA EXTENSION DIVISION
CORRESPONDENCE COURSES IN ENGINEERING SUBJECTS

Course 1A Elements of Surveying Swafford

Assignment 6

COMPASS SURVEYING

FOREWORD:

This assignment deals with compass surveying methods including traversing with the compass, computation of angles from bearings, bearings from angles, detection and correction of local attraction, and adjustments of the compass

(50) TRAVERSING WITH THE COMPASS

The compass was one of the earliest surveying devices used in modern surveying for determining the directions of lines, and while it is little used today except for reconnaissance and the checking of more accurate work, the method of traversing with this instrument should be thoroughly understood.

Traversing is the process of bounding a lot or field or measuring the angles and perimeter of the field. In other words, it is the process of traveling around the perimeter of a polygon in such a way as to determine its dimensions.

There are three common cases which arise in this work:-

1. That in which the actual corners of the fields may be occupied by the compass and in which the lines may be measured directly.

2. That in which the corners of the field cannot be occupied by the instruments, but in which the lengths of the lines can be measured directly.

3. That in which the corners of the fields cannot be occupied

to keep the rod plumb.

SURVEY OF WILSON'S FARM WITH COMPASS AND CHAIN.

Station	Obs. Mag. Bear. Forward	Obs. Mag. Bear. Reverse	Local Attraction	Corrected Bearings Forward	Distance Chains	Remarks
A			0°00'			
	N17°45'W	S17°45'E		N17°45'W	20.01	
B			0°00'			
	N72°15'E	S72°00'W		N72°15'E	9.99	
C			0°15' E			
	S54°50'E	N55°00'W		S54°35'E	24.73	
D			0°25'E			
	S71°50'W	N72°15'E		S72°15'W	25.00	
A			0°00'			
	N8°50'E	S8°35'W		N8°50'E	22.62	Check sight.
C			0°15'E			

≈ Interior Angles ≈

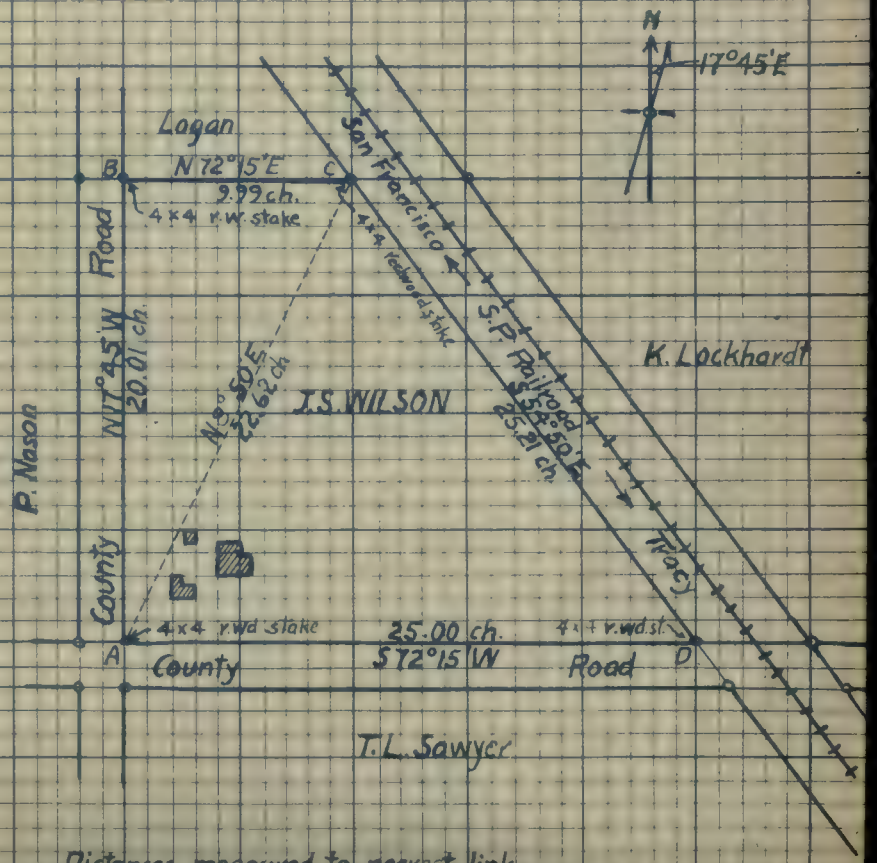
Course	Bearing	Angle
A-B	N17°45'W	
		90°00'
B-C	N72°15'E	
		126°50'
C-D	S54°35'E	
		53°10'
D-A	S72°15'W	
		90°00'
A-B	N17°45'W	
		360°00' ← Check

$2(n-2)\angle rt = 2(4-2) = 4\angle rt$
$= 360°00'$

Equipment:-
Lietz Compass.
2 Wooden Line Rods.
Gunter's Chain
11 Chaining Pins
3 Plumb Bobs

Nov. 16, 1915.
Clear and cold.

Jason, K.L. } Chain
Marcus, L.V. }
Larsen, J.N. }



Distances measured to nearest link.
Angles to nearest 5 minutes.

by the instrument and in which the lines cannot be measured directly.

The first case, which is the simplest, will be explained in this assignment. The second and third cases, which are not so easy of solution, will be considered under Traversing, Assignment X, Art. 94.

Refer to Figure 9, which represents complete field notes for a compass survey of a farm. The procedure was as follows:

Beginning at the point marked A the compass was set up over the stake and properly leveled. The instrument man then sighted at the point D and noted the bearing, N 72° 15' E, this value being recorded in the third column on the left-hand page of the notebook between D and A. This bearing is called the back or reverse bearing of the line DA or the bearing of the line AD. This terminology should be very carefully noted so that the student will not be confused in later discussions. The circle in the instrument used was subdivided into quarter degrees of 15' divisions but all readings were taken to the nearest 5 minutes by estimation.

After the above reading was recorded, the instrument man sighted on the point B and the forward bearing of the line AB or, simply, the bearing of the line AB was recorded in the second column on the line between A and B. A sight was then taken on the point C for the purpose of checking the work. This bearing was noted on the sketch on the right hand page.

The rodman held a rod over the points D and A being careful to keep the rod plumb.

The instrument man then moved to the points B, C, and D respectively, where he repeated the process as at A. At the point C a sight was taken to point A. The bearings read in each instance were recorded in their proper places in the fieldbook.

The distances were then chained and recorded in the sixth column on the left hand page.

The right hand page shows a sketch of the field which should always be a part of good field notes. Such a sketch should show the shape and dimensions of the survey approximately to scale, the position of important natural and artificial features, the name of the owner and the names of adjacent owners, the meridian (both true and magnetic if possible), and such explanatory notes as would make the findings clear to any later surveyor who might have occasion to use them. It is generally the experience of surveyors to find that most field notes are not full enough. The tabulated notes should tally with the quantities recorded on the similarly designated sketch and should be similarly designated.

The notes on the left hand page include a column for the stations occupied, a column each for the forward and reverse bearings of the lines, a column for local attraction, another for the corrected bearings, and one for the distances. The lower portion of the page shows the interior angles of the field and the usual check for such a survey.

(51) LOCAL ATTRACTION

Local attraction is the disturbance at any particular point

or station which causes the needle to swing out of the magnetic meridian. It is caused by the presence of magnetic metals nearby, or by electric power lines in the vicinity. The surveyor should take reasonable care that he does not carry iron keys or a knife, and that he does not leave any portable objects close to the instrument, such as chains, tapes, iron aligning rods, etc., that would disturb the needle. An annoying source of variable local attraction is found in steel buttons or clasps on clothing worn by the instrument man or his assistants.

Local attraction is appreciable over varying areas depending upon the magnitude of the source, but it is usually felt within a very restricted area and the disturbance decreases as the distance increases. The Law of Magnetic Attraction or Repulsion, is that magnetic attraction varies directly as the intensity and inversely as the square of the distance from the source.

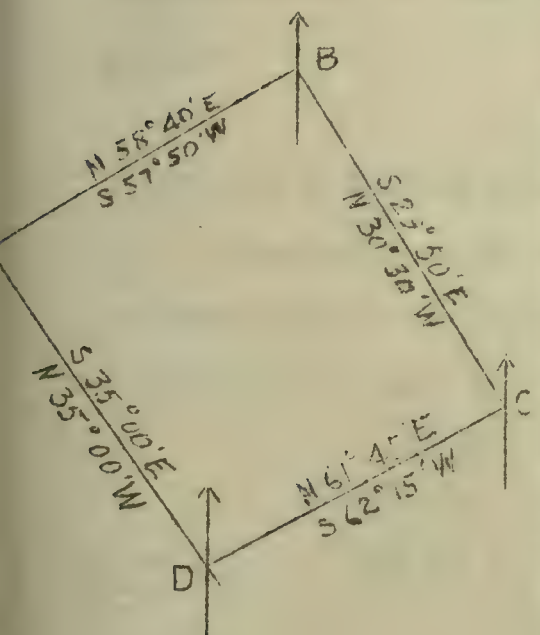
Where the local attraction cannot be removed the magnitude of the disturbance of the needle must be determined and corrections made in the notes.

(52) DETECTION AND CORRECTION OF LOCAL ATTRACTION

Again referring to Figure 9, columns 2 and 3, it will be seen that the forward and the reverse bearing of the line AB are just 180 degrees apart. This condition should hold for the two bearings of any line surveyed and would indicate that there was no disturbance of the needle at either of the two points. It might be, however, that the disturbance at both of the points was the same in intensity, but opposite in direction, although this would be exceptional.

(53) CORRECTION FOR LOCAL ATTRACTION

. If the forward and the back bearing check, that is, if they differ by 180°, it is reasonable to assume that there is no local attraction at either end of the line. It cannot be assumed, however, that, if they do not check, the local attraction is at either or both ends; this must be determined by taking bearings on other lines connecting with the two points. For example a closed traverse may be taken to illustrate a method of adjusting the local attraction. The following set of observed bearings, given both in sketch and tabulated form, will make this plain:-



Line	F. Bearing	B. Bearing	Correction	Adjusted Bearing
A-B	N. 58° 40' E		00	N 58° 40 E
B-A		S 57° 50' W	+ 50'	S 58° 40' W
B-C	S 29° 50 E		+ 20'	S 30° 10' E
C-B		N 30° 30' W	- 20'	N 30° 10' W
C-D	S 62° 15' W		- 30'	S 61° 45' W
D-C		N 61° 45' E	00	N 61° 45' E
D-A	S 35° 00' E		00	N 35° 00' W
A-D		S 35° 00' E	00	S 35° 00' E

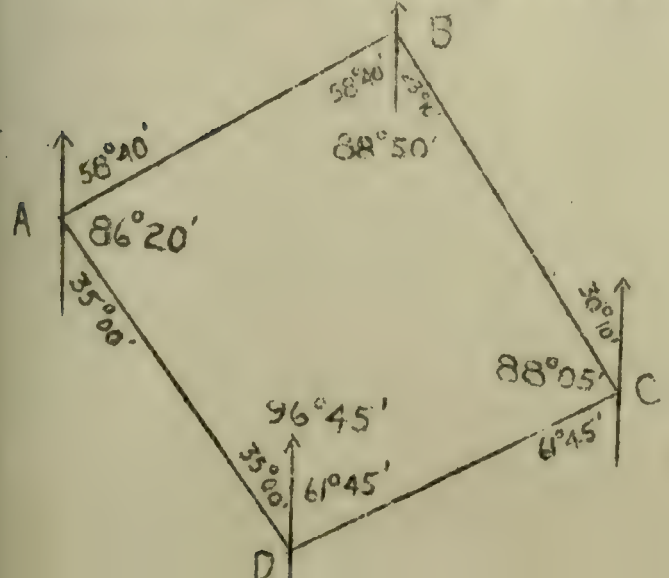
Figure 10.

At A and D it may be assumed, since the forward and back bearings check, that there is no local attraction; hence A - B is probably correct, N 58° 40' E; and B - A should be adjusted to read S 58° 40' W; likewise D - C is probably correct, N 61° 45' E; hence C - D should be adjusted to S 61° 45' W.

Evidently there is then local attraction from a source affecting the needle at both B and C. Here the forward and back bearings (B-C, C-B) differ by 40', and one half of this amount added to 29° 50' and also subtracted from 30° 50' will bring the bearings into agreement. Thus for this case the adjustments are made. The data is tabulated as shown above. The interior angles may now be computed and will be found to check, as they should (nearly) in practice.

54) COMPUTATION OF INTERIOR ANGLES FROM BEARINGS

By referring to the adjoining figure, the bearings being the adjusted values in the traverse just cited, it is easily observed that the angle ABC is the sum of the back bearing B-A and



Interior Angles

ABC	88° 50'
BCD	88° 05'
CDA	96° 45'
DAC	86° 20'
Total	360° 00'

PROB. NO. 3 TRAVERSE OF FIELD WITH COMPASS AND TAPE

Station	Obs. Mag. Bear. F.S.	Mag. Bear. B.S.	Observed Distance Ft.	Corrected Distance Ft.	Local Attraction	Adjusted Bearing
A					0°00'	
	S 30° 45' W	N 30° 45' E	249.3	249.3		S 30° 45' W
B					0°00'	
	S 85° 30' E	N 84° 30' W	375.7	375.7		S 85° 30' E
C					1°00' W	
	N 2° 00' W	S 2° 15' E	484.2	484.2		N 3° 00' W
D					0°45' W	
	S 89° 30' W	N 88° 45' E	352.6	352.6		S 88° 45' W
E					0°00'	
	S 28° 00' E	N 28° 00' W	257.8	257.8		S 28° 00' E
A						

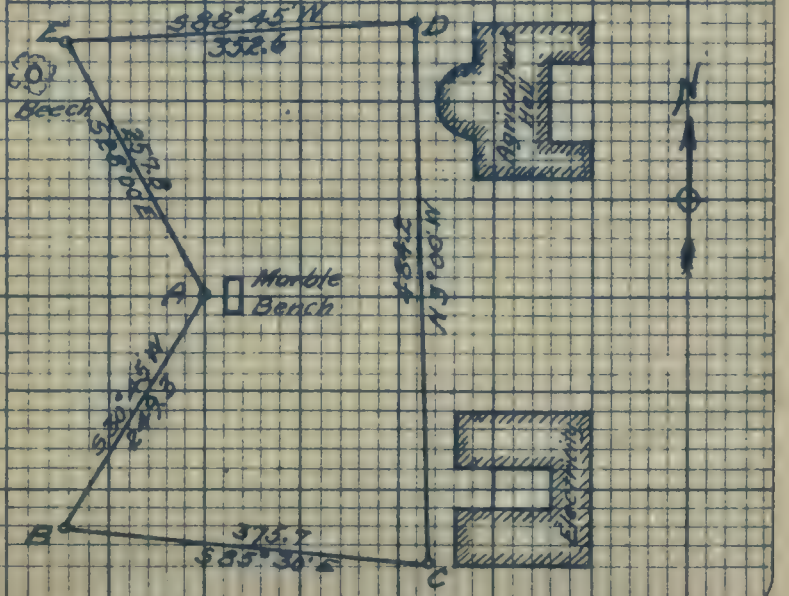
Interior Angles.

A	238° 45'	
B	63° 45'	n = 5
C	82° 30'	= 180°(n − 2) = 180° × 3 = 540°
D	91° 45'	
E	65° 15'	Check
	540° 00'	

Equipment:
Gurley compass #9837
100-foot steel tape #19
Chaining pins
2 Plumb-bobs
2 Line rods

Sept. 3, 1907.
Weather variable.
LaFevre, A. E.
Martin, J. L.
Wilson, J. F. } Chain.

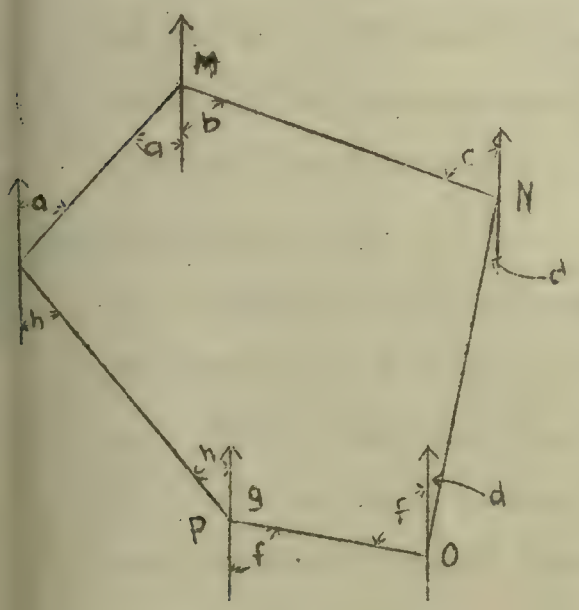
Determined length and bearing of each line of traverse. Lengths to nearest 0.1 foot bearings to nearest 15'.
Corrected bearings for local attraction.
Compared tape with University Standard.
Length of tape 100.00 feet.
Sketch shows reduced values.



the forward bearing B-C. Hence ∠ABC = 58°40' + 50°10' = 88°50'. Likewise, angle D is the sum of the back bearing D-C and the forward bearing D-A. Hence ∠CDA = 61°45' + 35°00' = 96°45'. The angle BAD is found by adding the forward bearing of A-B to the back bearing of A-B and subtracting this sum from 180°. 180° - (58°40' + 35°00') = 86° 20'. Also ∠BCD = 180° - (61°45' + 30°10') = 88° 05'.

Many rules are given by the various authors of texts on surveying, but by far the best and the simplest method is as follows:

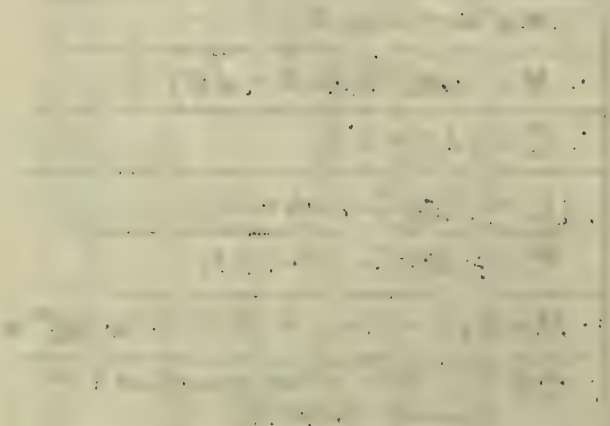
Sketch the meridian (magnetic or true) in accordance with the bearing) and determine the desired angle by geometric addition. A further illustration using general values for angles and bearings here follows:



Interior Angles

$M = \angle a + \angle b$
$N = 180° - (\angle c + \angle d)$
$O = \angle d + \angle f$
$L = 180° - (\angle a + \angle h)$
$P = 180° - \angle f + \angle h$
$M+N+O+P+L = 2(n-2) \times 90° =$
540°; this is the equation of correct condition.

Figure 12.



Check: The sum of the interior angles of any polygon is equal to twice as many right angles as the figure has sides, less four right angles. The figure here has five sides, Twice 5 = 10, from which subtract 4, giving a remainder 6. Therefore, a five sided figure gives 6 right angles or 540°. In general $S = (2n - 4)\ 90°$. This formula is also written $S = (n-2)180°$.

(55) TRAVERSING BY DEFLECTION ANGLES

Traversing by deflection angles is sometimes a convenient method, the angles both exterior and interior being easily calculated. If the deflection angles are measured around a field consecutively in either direction (either all right or all left deflection) these angles constitute the exterior angles of the polygon; their sum for any polygon is 360°. The interior angle at any vertex is 180° minus the exterior angle at that vertex; the sum of all the interior angles is 180° times the number of sides minus 360°, or in general $S = 180n - 360 = (2n-4)\ 90°$.

(56) RANGING OUT LINES

Ranging out is best done by observing the deflection angle and recording it as right (R) or left (L) at each change of bearing. This is especially true for a traverse that does not close, as a road, rail-road, boundary line, or ditch, the ranging of which consists of several parts of straight lines or lines and curves.

To measure a deflection angle, set up the instrument at the forward end of a line and back-sight on the other end. Then

placing the eye at the opposite sight, turn the line of sight right or left as the case may be and read the angle passed over by the line of sight when bisecting the next point in order. As follows:

To range out the line O A B C D E by deflection set the instrument at A and back-sight on O. Then turn fore-sight on B and

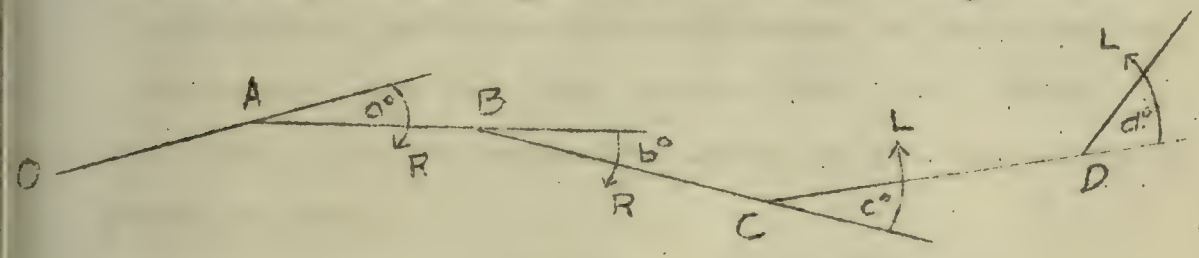


Fig 13

read a° R. Next occupy B, and back-sight on A; turn fore-sight on C and read b° R. Then at C read c° L; at D, d° L. The angle A is 180° - a°. B is 180° - b°, etc.

The direction of lines, or their bearings are often given in azimuth. The azimuth of a line is its bearing measured in degrees in clockwise direction from either the north or south point of the horizon; either the magnetic or the true north (or south) may be chosen as zero azimuth. This method will be more fully explained later, under transit surveying.

REFERENCES:

Tracy, pp. 378 - 383

Breed & Hosmer, pp. 29 - 30, Vol. I.

Johnson, pp. 34 - 38.

Raymond, pp. 83 - 85.

PROBLEMS

1. The courses and bearings of a certain traverse are as follows: A-B, S 35° 32' E; B-C, N 71°34' E; C-D, N 18°26' E; D-E, due West; E-A, S 41°09' W. Find the interior angles and the error of closure. Make a sketch of the field.

2. A triangular field was surveyed with compass and the deflection angles were taken as follows: at A-B, 93°15'; B-C, 150°30'; C-A, 116°11'. Compute the interior angles, and give the discrepancy. Sketch the field.

3. A certain survey recorded magnetic bearings as follows: 1-2, N 43°15' E; 2-3, S 73°18' E; 3-4, S 83°23' W; 4-1, N 80°17' E. If the magnetic declination at the time was 12°15' E, what was the true bearing of each line?

UNIVERSITY OF CALIFORNIA EXTENSION DIVISION

Correspondence Courses

Surveying--1A Elements of Surveying Swafford

Assignment 7

THE LEVEL AND ITS USE

FOREWORD:

This assignment will deal with the level, one of the most important instruments used by the surveyor. Many of the principles treated in this part of the course are fundamental. You are urged to master the subject of leveling, not omitting the slightest detail, because much that follows in the subject of surveying depends upon the material given here.

57) THE LEVEL

In its usual form, the level is an instrument for determining the heights (elevations) of points, lines, and surfaces from some known surface. It consists of a spirit level and a telescope, so combined and adjusted as to enable us to determine elevations.

The illustration, Plate I, shows a plain type of surveyor's level, which will be fully explained in a subsequent paragraph in this lecture. For the present let us turn attention to a few definitions of fundamental importance.

A level surface is one that is everywhere perpendicular to the plumb-line drawn to a point on that surface. Such a surface is not a plane, but is curved in all directions. As it will approach very nearly a spherical surface, for the present purpose we will speak of it as such.

The surface of a body of water at rest is level; and a large body of water, such as a lake or ocean, is known to be curved. In the more exact work of leveling, the surveyor actually measures and determines the curvature; in the less exact work he determines only a tangent plane or several tangent planes which suffice for the purpose in hand.

A level line is any element of the level (curved) surface. A plane can have only straight lines lying in its surface, a plane tangent to a level surface meets that surface in only one point, the intersection of the plumb-line at that point. Hence a straight line and a level line coincide only in that point (called point of tangency) where a line drawn directly to the earth's center meets both lines.

The points along any vertical line may have surfaces passed through them and thus a vast number of concentric spherical surfaces may be found. These surfaces are designated by the heights of a point in each above (or below) a certain agreed surface called Datum or datum level.

Datum is generally taken to be mean sea level, although any other surface may be assumed for reference. Points, lines, or surfaces referred to the datum surface are said to be so much above (or below) datum, the distance being given in the usual linear units. Since it is not always practicable to use mean sea level as datum, other reference planes or surfaces may be selected. For most purposes these are just as good; but where the established

levels are to be of a permanent and extensive character, sea-level is the best reference plane and should be studiously sought.

(5) THE SPIRIT LEVEL

The spirit level is a simple instrument consisting of a tightly sealed glass tube, or phial, almost filled with liquid, water, alcohol, or ether, imprisoning a small quantity of air in the part not filled with liquid. This air constitutes the bubble as it is called, on which observation is made in the use of the spirit level. The tube with its contents of liquid and air is generally briefly called "bubole tube". The bubble-tube is slightly curved, so that when placed longitudinally in a horizontal posture, the convex surface is uppermost. Thus the bubble of enclosed air is forced by the many times heavier liquid to the summit of this curve, and thus caused to occupy a fixed position for a given horizontal one of the containing tube. Just as the plumb-bob and its cord enable one to determine a vertical line, so the bubble-tube may determine for us a horizontal or level line; and by the combined use of plumb-line and bubble-tube, many desirable quantities may be measured, lines located, and relations established. The spirit level enters into the construction of many useful instruments and in its use becomes the surveyor's constant reliance for determining horizontality and perpendicularity. The compass, described in Assignment 5, usually carries two bubble-tubes. The spirit level is an essential part of the engineer's level, the surveyor's or engineer's transit, and the plane-table alidade. It is

also used for leveling tapes, for plumbing rods, etc.

The spirit-level is sometimes made in a circular form that is very convenient for some purposes; for example, for leveling the plane-table and for plumbing rods. The upper, curving surface of the circular level, made of glass, is a portion of a spherical surface, the sphere being of great radius as compared with the size of the level. The imprisoned air, which is the bubble in this form as in the tube form, rises to the highest part of the glass cover, and so the bubble is made to occupy the center of the circle when the plane of the level is horizontal. This accomplishes all that can ordinarily be accomplished by means of two bubble-tubes placed at right angles to each other; a level plane may be determined by the single circular level, which is often of great advantage.

) PRINCIPLES OF THE ENGINEER'S LEVEL

1. If a horizontal line be made to rotate about a point in itself, but always maintaining a horizontal position, it will generate a horizontal plane.

2. Straight lines lying in this plane will be horizontal lines.

3. All lines passing through the point about which the given line rotates will be perpendicular to a plumb-line at that point.

In the engineer's level the rotating line is <u>the line of collimation</u> of a telescope which may be prolonged as the <u>line of sight</u> to an indefinite distance.

This line of collimation is made perpendicular to a plumb-line, called the vertical axis of the instrument, about which the telescope rotates, therefore, in a horizontal plane.

Conceive then this line determined by the telescope's axis sweeping out a horizontal plane tangent to the true (spherical) level surface at the point of intersection with that surface of a plumb-line, the vertical axis of the instrument. The height of this plane measured along the plumb-line (vertical axis) from some assumed plane of reference (datum plane) is called the height of instrument (H. I.)

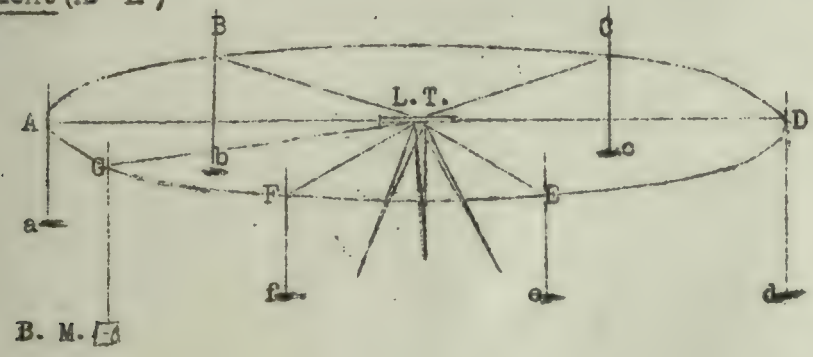


Figure 14

In the illustration (Fig. 14) L.T. is a level telescope supported upon a tripod that is in adjustment, carefully set, and "leveled up" with the tripod legs firmly planted in well chosen positions in the midst of a number of points at the surface of the ground, a, b, c, d, e, f, and also an established point B.M. (bench mark) shown to the left. The telescope rotated about its vertical axis will determine the horizontal plane A B C D E F G. If now we measure the heights of the horizontal plane above the points on the ground we obtain the quantities aA, bB, cC, dD, eE, fF and (B.M.) G. Now the height of B.M. above the plane of reference, called datum level, is known. Hold a rod on B.M. and direct the line of sight toward it and read the distance (B.M.)G. Add

PLANE SURVEYING

COURSE X–1A

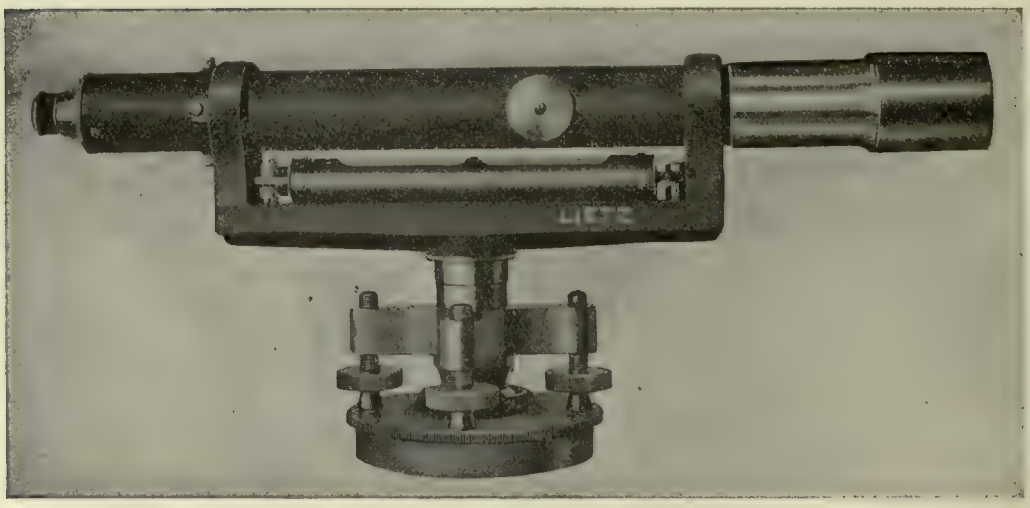


PLATE I

16-INCH DUMPY LEVEL

Length of telescope, 16″; diameter of objective, 1⅜″; magnifying power, 28×; total weight, 25 lbs.

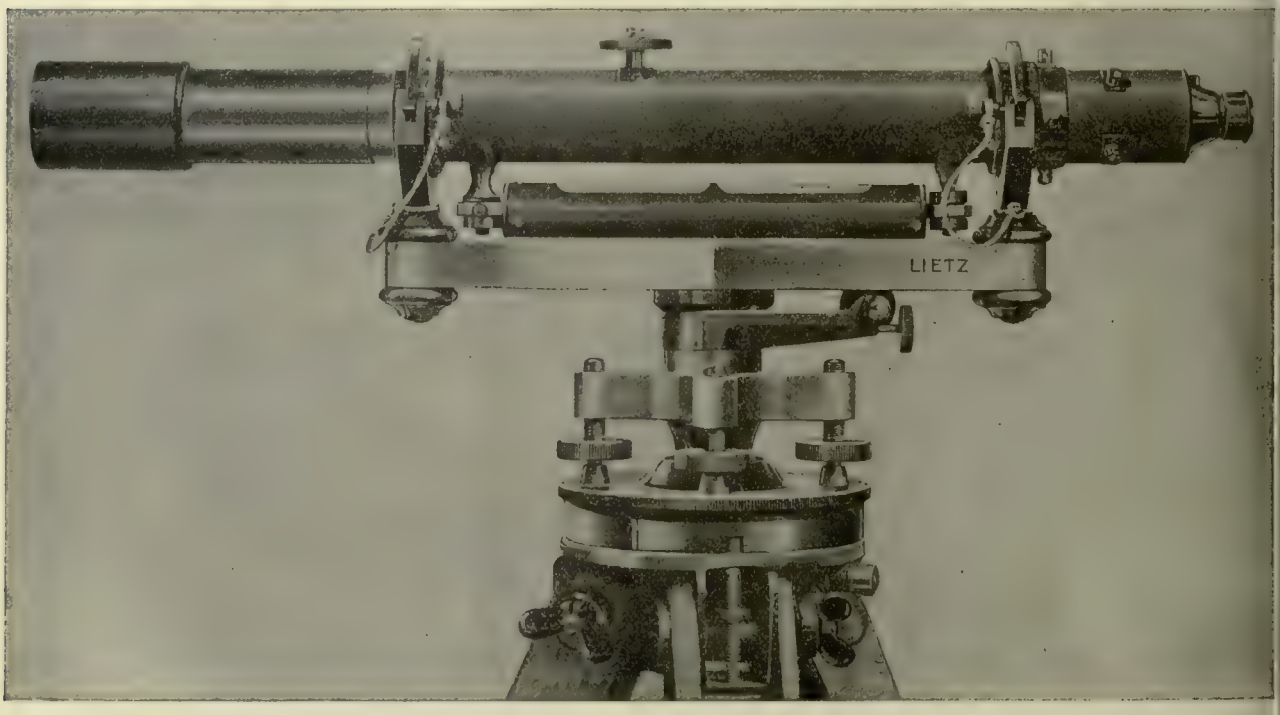


PLATE II

PRECISION ENGINEERS' Y-LEVEL

Length of telescope, 18″; diameter of objective, 1⅜″; magnifying power, 33×; total weight, 27½ lbs.

this reading of the rod to the height of B.M. which will give the height of the level plane above datum. This is called the height of instrument, H.I. The plane ABCDEF is at the height H.I. above Datum.

To find the height of a, b, c, etc. above datum, hold a rod (level-rod) at a, b, c, etc. and direct the line of sight to it and read the heights aA, bB, cC, etc. Subtract these readings from H.I. and the differences will be the elevations of the points on the ground above Datum.

Note well that the rod-reading on B.M. giving the quantity (B.M.)G is added to the height of B.M. to give the H.I., and that the rod-readings aA, bB, cC, etc. are subtracted from H.I. to give the elevation of the points on the ground. Hence, since the B.M. reading is added, we regard it always as a + quantity, called a plus sight, and, for a like reason, the point whose elevation is to be determined we regard as a - quantity, called a minus sight.

The terms Back-sight for plus sight and Fore-sight for minus sight are also commonly used. You should get the foregoing conceptions clearly in mind, as much confusion is often occasioned by a vague understanding of these very simple matters.

The following is a brief description of the engineer's level. For simplicity the Dumpy level is chosen. The student should procure a descriptive catalogue or manual of surveying instruments such as is published and for sale at small cost by the

leading instrument makers - L.C. Berger & Sons, Buff and Buff, W. & L. E. Gurley, Bausch & Lomb, The A. Lietz Co., and others. The manuals go much beyond the text-book on the subject, and usually give a deal of information, tables, and material of great value to student and engineer alike. The best way to know the instruments is by intelligent handling of them guided by a skilled instrument man or by following directions given in good text-books or the manuals named above.

C) THE DUMPY LEVEL

As shown in the illustration, Plate I, the Dumpy Level consists of a horizontal bar fixed to a vertical axis which turns freely in a well made socket. To the bar a telescope and a spirit level are firmly attached. The whole is supported upon a tripod and a leveling head. The latter is used to set the bubble in a central position in its tube, thus bringing the line of collimation of the telescope (which should be parallel to the axis of the bubble tube) into a horizontal position. The line of collimation can then be made to sweep out a horizontal plane by turning it upon the vertical axis of the instrument, prolonged indefinitely in the line of sight.

The telescope contains two optical parts, the object-glass at the larger end and the eye piece at the smaller end of the telescope tube. The object-glass is a double-convex lens usually of the compound variety to overcome the tendency to color dispersion of light by unequal refraction, called chromatic aberration. This lens is also shaped in a manner to reduce as far as possible a

distortion known as spherical aberration. Both of these faults of simple lenses, would be very troublesome, if the skill of the instrument maker could not combine and construct lenses so as practically to eliminate aberration. For a full explanation of these matters you should consult a reliable work on optics; in general, a good high-school text on physics will be sufficient.

When the light from the various parts of any object to which the large end of the telescope is directed passes through the double-convex lens, it is made to converge within the tube in such a way as to form a real but diminutive image of the object sighted. This real image is inverted. If now, the real image be viewed by another lens, the eye-piece, it will be magnified so that its details will be more distinct and it will appear nearer and also larger. In some instruments a compound eye-piece is employed, which re-inverts the image that was inverted by the object-glass; thus the thing viewed through the telescope appears in its normal position or erect. On this account a telescope with the compound eye-piece is called an erecting instrument. The sort having a simple (single) lens for eye-piece is an inverting instrument.

The inverting type is preferable for many optical reasons; and when once the engineer becomes accustomed by long use to the inverted images, he may prefer an inverting to an erecting instrument. The inverted image is brighter, and more sharply defined, and it gives a flatter field, all of which are very desirable. However, it is not intended to disparage the erecting eye-piece, as

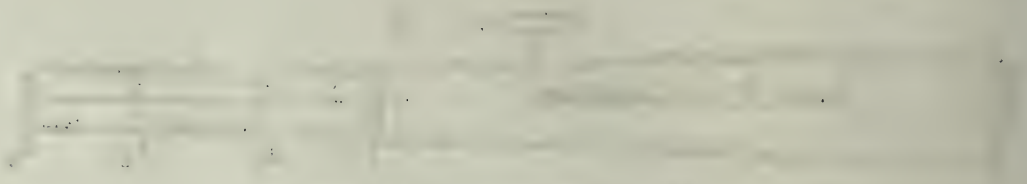
all makers construct excellent instruments of this latter kind, and many engineers prefer to use them. Within the limits of ordinary surveying the erecting instrument is quite adequate and very convenient.

Section of Erecting Telescope
Figure 15

The accompanying cut shows a section of the telescope. O is the objective lens and 1, 1_1, 1_2, 1_3 are a combination of lenses constituting the eye-piece. The lenses are named in order, 1, the object lens; 1_1, the amplifier; 1_2, the field lens; 1_3 the eye-lens. Together they form a compound magnifier (microscope) which enlarges and re-inverts the real image formed by the objective-lens, O. This real image falls at C, at which is placed a ring-like diaphragm carrying two fine threads, either of platinum or spider web, called cross-hairs or cross-wires. Thus the image viewed through the telescope has the image of the cross-hairs lying distinctly upon it The line joining the intersection of the cross-hairs and the optical-center of the objective-lens with the optical-axis of the telescope, when in coincidence form the "Line of Collimation" to which reference has several times been made.

(61) THE WYE-LEVEL

Another common form of engineer's level is the Wye Level shown in Plate II, in which the horizontal bar, instead of having



the telescope attached to it, carries at each end a clutch for holding the telescope called a Wye (or Y) since it resembles the letter Y.

The wyes are accurately ground to shape on their supporting surfaces in which rest two collars forming a fixed part of the telescope-tube, The collars also are very accurately ground to cylindrical shape and of equal diameter, so that, when resting in the supporting surfaces of the wyes, the optical axis of the telescope (the line of collimation) is perpendicular to the vertical axis in whichever of its two postures it may lie. Thus the telescope may be turned end for end in the wyes, or rotated about its axial line when resting in it. Each wye has fitted to it a cover-clip that fastens down either with a pin or other device. In one collar (usually the eye-end) is a notch into which a pin on the clip fits, thus holding the tube firmly and preventing its rotating when the clip and pin are down.

The bubble-tube in the wye-level is fastened by short column-like supports to the telescope-tube so that it may be adjusted to be parallel to and lie in the same plane (i.e. truly parallel) with the line of collimation of the telescope. See Assignment XIII on Adjustment of Instruments.

The wye-level has long been in favor with engineers on account of the readiness with which certain instrumental adjustments may be made, and perhaps on account of the belief that it embodies some features thought to be indispensable in an instrument of presicion. These features are, however, rather more fanciful and

traditional than real or even convenient. The dumpy-level is coming more and more in favor every day; and for precise leveling other and unquestionably better forms have superseded the wye-level. Still the wye is by no means an obsolete pattern, and is still preferred by many surveyors. You are, therefore, particularly advised to acquaint yourself with the structure, adjustment, and use of this instrument.

(62) THE HAND-LEVEL

The hand-level is a simple and very useful leveling tool. It is specially adapted to reconnaissance and preliminary work, or to levels where a low degree of accuracy is permissible.

The Locke hand-level is the simplest and most commonly used type, but others of a more refined and elaborate construction are also made by most manufacturers of instruments. They are described in catalogues and manuals. The Abney hand-level and clinometer, and the Atwood combined hand-level, clinometer, and compass (a really universal instrument) are the principal refined forms of this useful little tool.

The very elaborate precise-levels, such as are employed in the Coast and Geodetic Surveys and other extensive leveling work, are not in general use by engineers and will not be described in the present course. The manuals referred to furnish illustrations and descriptions for those who desire to know more about them.

The level in any of its forms is used to determine heights

or elevations from an assumed datum, to measure differences of elevations or altitude, to run profiles, lay off cross-sections for fill and excavation, locate contours, etc., etc. The work of such nature will be explained and amplified in subsequent assignments, which will deal specifically with each class of problems.

QUESTIONS

1. What do you understand by H.I.?
2. Explain B.S., F.S., B.M.
3. Why is a level line not a straight line?
4. A level set up on the shore of a lake had its line of sight in a plane 6.5 feet above the water in the lake. The readings taken on a rod held at various points gave at A 4.3 ft., at B 8.7 ft., at C 6.5 ft.; how do the heights of these points compare with the level of the water in the lake?

References:

Tracy, Chapters XIX and XX.
Raymond, Ch. III.
Johnson, pp. 55 - 56, 60 - 64.
Breed & Hosmer, pp. 72 - 78, Vol. I.

UNIVERSITY OF CALIFORNIA EXTENSION DIVISION
Correspondence Courses

Surveying-1A Elements of Surveying Swafford

Assignment 8

LEVELING PROBLEMS

FOREWORD:

This and the following assignment will describe the use of the engineer's level (A) for determining Difference in Elevation; (B) for Profile Leveling; (C) for Establishing Bench Marks; (D) for Cross-section Leveling; (E) for Leveling for (1) Excavation (Borrow Pits) and (2) Fills; and (F) for Finding Grades and Contours. The methods of making notes for the various problems here treated will also be shown.

(63) GENERAL CONSIDERATIONS

The level should always be set up on firm ground at a convenient station for observing as many points and as large a range as is consistent with good work within the limits of accuracy sought. It is rarely necessary to set the level over a point, the elevation of which is desired, or immediately upon a line joining two points to be measured. Indeed, such a setting will often frustrate the purpose in hand or greatly interfere with accuracy in observation, as will be perceived if you recur to the conception of a horizontal plane at elevation H.I.

(64) BENCH MARKS

If elevations of points are desired above any agreed plane of reference (or datum level), a bench mark (B.M.) of known elevation must begin or close the work.

Should the extent of the leveling require that the level be moved from place to place, points of firm and often of durable nature must be selected or improvised on which an observation (fore-sight) may be made at one setting and another observation (back - sight) may be made at the following setting. These are called Turning Points, perhaps because the measurements are made from a new horizontal plane swept out by the line of sight at each setting.

(65) Turning points are usually selected or established as the work of leveling proceeds, and much depends upon their kind. They should always be sufficiently firm to support the rod held upon them while both a fore-sight and a back-sight can be taken upon them. They should be situated in open spaces where both sights may be clearly taken free from obstruction, not too near or yet too remote from the instrument. The distances from two successive turning points (T.P's) to the instrument should be about equal. This last requirement is for the purpose of eliminating any error in the parallelism of the axis of the bubble-tube with the line of collimation of the telescope. For, if the vertical axis be truly vertical, but the line of sight for any reason be not perpendicular to it, then, when the bubble is at mid-point of its tube for each sighting on two points equidistant from the instrument, these two points will be in the same level plane. The line of sight in such case "sweeps out" the surface of a cone and the vertical axis of the instrument is also the axis of this cone. The position of the cone is either erect, standing upon its base, if the line of sight is inclined downward, or inverted, standing upon its apex, if the

line of sight is inclined upward. Now, since the two points are at equal distances from the instrument (i.e. from the vertex of the cone) and the axis of the cone is vertical, a horizontal plane perpendicular to the vertical axis may be made to pass through both points. Hence they are at the same elevation and a line joining them is a level line.

(66) THE LEVEL ROD

For the purpose of making vertical measurement a rod is used, on which is marked off a scale of linear measure, feet and inches, or better, feet and decimals of feet, or in other cases, meters and fractions of meters.

The common form is in two parts, each about 7 1/2 feet long, 1 1/2 inches wide, 3/4 inches thick, graduated upward upon the front side and continuing downward on the reverse side of each part. The parts are fitted together and are held in place by two bronze sleeves. The upper sleeve, turned toward the back, has a vernier scale on its edge, by which the least interval capable of being measured may be determined. The upper sleeve also carries a clamp screw, with a large milled head, by means of which the rod, when extended, may be held firmly in position. Thus the rod may be used either short (up to 7 feet) or long (to about 13 1/2 feet when fully extended). The lower end of the rod is fitted with a metallic shoe.

(67) A TARGET

The target, made of metal in circular or other convenient shape, about 5 inches in diameter, has an oblong opening in its

middle part. It is suitably fitted to slide up and down upon the front section of the rod. On one edge of the opening a second vernier is attached for reading the least measures as with the first vernier. The face of the target is divided into four equal quadrants which are colored alternately white and red, their horizontal line of division being coincident with the zero of the vernier scale. A clamp-screw similar to the one mentioned above is carried by the target, by means of which the target may be set at any point along the front section of the rod. It may also be fixed at the 7 foot mark of the rod on a segment of the front section that is firmly glued and screwed upon the rear section. The best rods are made of well seasoned, straight-grained maple wood, neatly fashioned and highly finished, and the divisions are accurately and distinctly marked in black and red upon a white ground.

(o8) THE VERNIERS

The above mentioned verniers are constructed as follows: For a direct-reading vernier, a space equal to 9 of the smallest divisions on the rod is divided into 10 equal parts on the vernier. Let us take the common graduation in feet, tenths of feet, and (the least division on such a rod) hundredths of feet. Then in the vernier for this graduation (Direct type) will consist of 9/100 feet divided into 10 equal parts, and each vernier interval will be 9/1000 feet. If now the horizontal line of the target, which coincides with the zero or lowest line of the vernier divisions, be set at any point on the rod, say at 4 ft., 3 tenths, and 6 hundredths and some fraction above the 6 hundredths mark, the vernier

will determine for us the number of thousandths beyond the 6 hundredths mark. For example, with the setting of target as given above, 4.36+ ft., count the vernier divisions beyond the 6 mark to that vernier division which coincides with a hundredth division line of the rod. This will give the number of thousandths of a foot to be appended to the reading above. For instance, if the number be 7 vernier divisions, the exact full rod-reading will be 4.367 ft. And so for any other cases.

In reading a direct vernier always count along forward in the direction of the main rod scale and add this count to the reading upon the rod as shown by the last full division. This will be upward on the front section of the rod, and downward on the reverse side of the rod when using the rod extended.

Generally, and for good reasons, a better form of vernier, called the Retrograde Vernier, is now employed. This is constructed by taking a space equal to eleven least rod divisions and subdividing it into 10 equal parts. Thus one division of the vernier is (on the retrograde vernier) 11/10 of one hundredth of a foot. So if the zero of the vernier scale fall at a point beyond a full hundredth division on the rod-scale and the vernier scale be applied in a reverse (retrograde) direction to the scale divisions on the rod, the count on the vernier scale backward to the line of coincidence will measure the number of thousandths of a foot beyond the last full hundredth mark. Hence read the front of the rod upward as always, but read the retrograde vernier downward; likewise on the

reverse scale for long rod, read the rod divisions downward and count on the vernier scale upward. The following illustration shows settings for both direct and retrograde verniers for both short-rod and long-rod readings. You are advised strongly to make many similar settings and record the readings for a rod to which you may have access. Check back the readings recorded, by resetting the target for each of the whole series of observations.

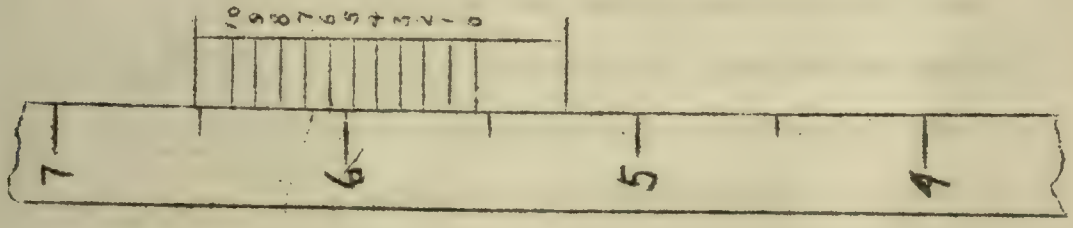
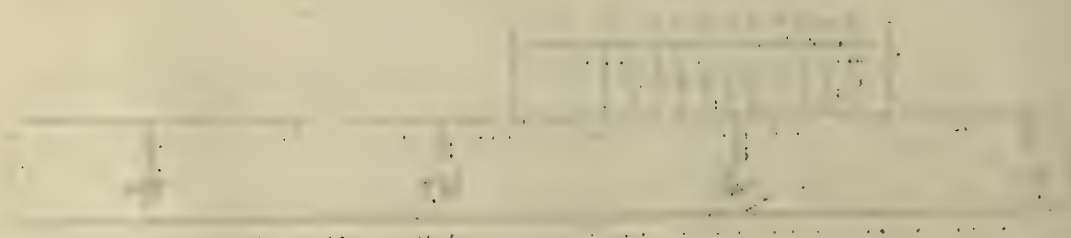


Figure 16

(69) USE OF THE LEVEL ROD

When taking readings on a target rod, be sure to close the rod down to its shortest length for short-rod, or set the target exactly on the highest mark for long-rod. Also be careful that nothing such as mud, gravel, or other foreign matter intrudes itself between the shoe and the point (B.M., T.P., or other surface) the height of which is sought. This could cause an error of several thousandths or even hundredths of feet, thus voiding the accuracy of the measurements. A proper bench mark is usually free from foreign matter or can easily be rendered so. Turning points should be selected upon a stone firmly imbedded in the ground, the root of a tree, or a stump or post, a curb-stone,



doorsill, etc.; T.P.'s may also be put down in the form of wooden stakes or metal pins driven well into the ground. A good form of T.P. is easily made of a triangular piece of light plate iron, which has a round head rivet set in its center and the corners turned down sharply at right angles to the plate, as shown in the figure.

Fig. 17

When such a turning point is pressed down into the earth, especially in light, marshy, or sandy soil, the rod can be held upon the rivet head at the center, thus insuring an excellent support. If a hole is punched in the plate so that a cord or chain may be attached, it can easily be lifted up and carried from point to point. Be careful also that the T.P. is even and horizontal, for if not, the rod might be too high in one position, and too low in another.

(70) THE USE OF THE TARGET

In leveling, the use of the target greatly facilitates the work of accurate sighting and at long ranges, several hundred feet distant, it is nearly if not quite indispensable to good work. Several forms of targets are in use, but the circular with quadrantal divisions is perhaps the best. The angular target which is in reality two half ellipses viewed at an angle of 45 degrees and thus appearing as a circular disk, is much in favor with many engineers. Such a target presents a sharply defined vertical edge

(the intersection of the two semi-ellipses), which assists the instrument man in determining whether or not the rod is in the vertical plane through his station; and the straightness of two horizontal lines, continuous as one line, enables him to know that the rod is also in the vertical plane at right angles to the first plane, or, in other words, plumb. For, if the rod be in the first vertical plane the vertical cross-hair will coincide with the intersection, and if not in the second vertical plane, the horizontal divisional lines will form either a wide vee ( ) when the rod leans toward the instrument, or will form a wide inverted vee ( ) when it leans backward, as shown by the horizontal cross-hair.

(71) PLUMB ROD, ROD LEVELS

It is very important in level work that the rod be held plumb when a reading is taken, as any deviation from perpendicularity to the level surface will give a false height, and the true height is the thing sought.

Rod Levels are frequently used, especially in nice work. These are either two small spirit-level tubes encased in a bronze block that fits squarely upon a corner of the rod, or a single level set in a block that may be applied by the rod-man when required.

The circular or bullseye level is also used for this purpose. It is supported upon the rod by a bracket that secures the true position. A plumb-bob and line may also be used. The rod-man

can also often secure the rod's verticality by aligning the same by eye with convenient structures near at hand or in the not too remote distance.

Still another way is to place the rod in the vertical plane through the instrument as nearly as possible, which is the less difficult feat of the two; then at a signal from the instrument man to "wave rod", he may slowly tilt the rod forward and backward while the man at instrument determines by trial the highest position of the horizontal line of the target. This last is the least satisfactory method, but in the absence of other, better means, it should not be neglected.

(72) CHOOSING SETTINGS AND T. P.'s.

In moving from setting to setting of the level, or in selecting turning points, it is essential that both rod-man and instrument man use careful judgment in each case. Often, after much trouble in choosing such positions, they are found unsatisfactory from the fact that the level is either too high or too low. This is especially liable to be the case on very uneven ground or in regions where rocks and trees or brush render sight difficult or impossible. At times the levelers must "hasten slowly", or at least exercise much ingenuity and good judgment. Often the longest rod is not long enough or the level plane at H. I. cuts the ground below the foot of the rod. Nothing can then be done but either to set up the instrument anew or to select a more suitable turning point, if it is this that makes the difficulty.

By roughly sighting along the barrel of the telescope when approximately adjusted to level, an advantageous setting may often be quickly determined.

It has been the purpose of the author to give detailed directions, as the student of this course is supposed to do all instrument and field work unassisted by an instructor to direct his operations. Much must always remain vague until by experience in handling instruments and laying out work in the field, indistinct, misunderstood, or erroneous mental images (such as one may acquire from books of description and illustration) are cleared up and corrected by actual practice. Therefore, it is with earnest sincerity that you are again urged to delve into texts and manuals on surveying in order that you may secure an intimate acquaintance with both instruments and surveying methods. The references at the end of each chapter to standard works will be of much value to those who have access to and will properly use such books.

(73) GENERAL PROBLEM IN LEVELING. A - Difference in Elevation

The problem is to determine the difference in elevation between two points A and B.

Set up the level at any convenient place about equidistant from the two points to be observed and follow directions, observing the cautions given in the preceding part of this lecture. Level the instrument, reversing over each diagonally opposite pair of foot screws. The rod-man must now hold the rod upon the point A in a vertical position with target out of range of line of sight

so that the instrument man may read through the telescope the approximate height of the horizontal plane. Then call to or signal this number (rod-reading) to the rod-man, who will quickly set the target as directed, but hold it if within comfortable reach, and move it up or down until signalled to clamp it. At the signal "wave rod", he will tilt it forward and backward. Determine whether the target is at the highest point; that is, whether the line of sight is never below the horizontal line of the middle of the target and whether it cuts this line but once at each forward or backward movement of the rod when it is "waved". This is the correct target setting, which the rodman now reads for himself, and, in passing the instrument to occupy the point B, the instrument man also reads the setting and records it in his field book when it is found to agree with the reading determined by the rodman. He calls "check" if correct; otherwise the rod should again be held upon the point and re-read to insure absolute agreement. At point B the same process is repeated and the elevation of B taken - that is, the rod in both cases has measured the distances from the points A and B to the level-plane swept out by the line of sight, or what is the same thing, the distance of the two points below this level-plane. The difference between the two rod readings is the difference in elevation sought. If the two points A and B are very distant from each other, or if the intervening ground is very sloping, or if the two points can not both be observed from a convenient setting of the instrument, it becomes necessary to select one or more turning points and to observe on these, finally closing on the second given point.

PROB. NO. 4 DIFFERENTIAL LEVELING WITH HAND LEVEL

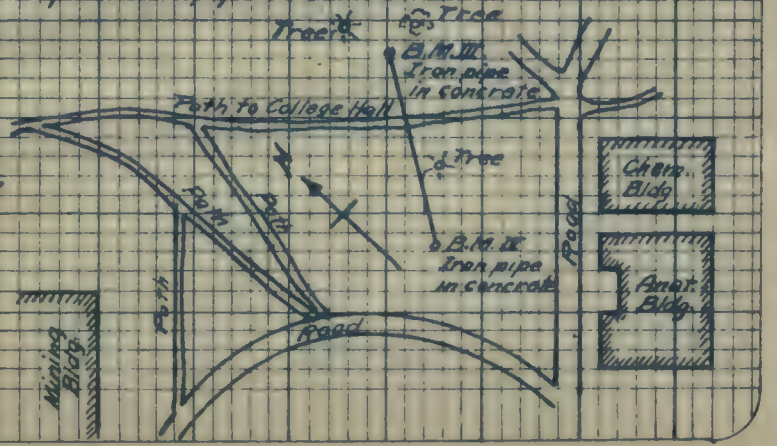
Station	+ B.S.	H.I.	− F.S.	Elev.	Diff. in Elev.	
B.M. II	10.5	380.2		369.7		
T.P. #1	11.8	391.5	0.5	379.7		
T.P. #2	9.3	399.1	1.7	389.8		
T.P. #3	7.6	405.9	0.8	398.3		
B.M. III			1.2	404.7		
	+39.2		−4.2	Check		
	−4.2					
	+35.0	+369.7	−404.7		35.0	
B.M. III	0.9	405.6		404.7		
T.P. #A	1.4	396.3	10.7	394.9		
T.P. #B	3.1	389.5	9.9	386.4		
T.P. #C	0.4	377.9	12.0	377.5		
B.M. IV			8.6	369.3		
	+5.8		−41.2	Check		
	−41.2					
	−35.4	+404.7	−369.3		35.4	
					0.4	Discrepancy
					35.2	Mean difference in elevation

Equipment:
K. & E. hand level #109.
Level rod #16.
Level support.

Sept. 10, 1902
Clear and cool.
Jones, G.C. & Miller, N.V. C

Top of iron pipe in concrete.
Highest point on large, imbedded stone.
Hub.
Center of path to College Hall.
Top of iron pipe in concrete.

Top of iron pipe in concrete.
Center of old stump.
Top of lath stake.
Edge of wooden curbing.
Top of iron pipe in concrete.



To illustrate this and to furnish a correct form of notes, take the following: First setting (1) Rod-reading on A 4.376 ft.; (2) rod-reading on T.P.$_1$, 6.843 ft.; remove level to second setting; (3) rod-reading on T.P.$_1$, 3.572 ft.; (4) rod-reading on T.P.$_2$, 5.831 ft.; third setting of instrument; (5) rod-reading on T.P.$_2$, 2.743 ft.; (6) rod-reading on T.P.$_3$, 6.745 ft.; fourth setting; (7) rod-reading on T.P.$_3$, (on the long-rod, i.e., rod extended with the target clamped at the topmost graduation; note that this reading is made on the back part of the rod and downward), 9.357 ft.; (8) rod-reading on B, 3.852 ft., which completes the course from A to B through three T.P's. We now proceed to tabulate these in notebook form as follows:

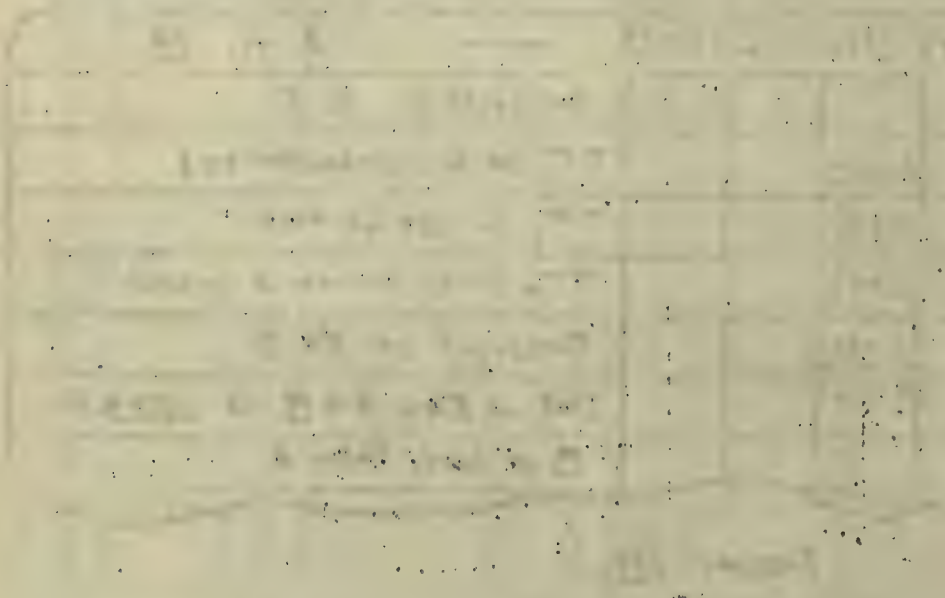
Differential Levels — A to B

Sta.	B.S.		F.S.			
A	4.376					Backsight on Sta. A.
T.P.$_1$	3.572		6.843			T.P.$_1$ on firm imbedded rock.
T.P.$_2$	2.743		5.831			T.P.$_2$ on root of tree.
T.P.$_3$	9.357		6.745			T.P.$_3$ stake driven in ground.
B			3.852			Foresight on sta. B
	20.048		23.271			Diff. in Elev. A + B is 3.223 ft.
	-23.271					
	-3.223					B is lower than A.

Figure 18.

The illustration is intended to show two pages of Field Note-book of approved form.

Note:- It is best, when the difference in elevation should be measured with precision that a series of return levels be run

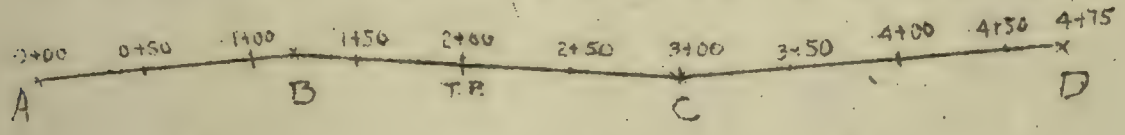


beginning upon B and closing upon A. The T.P's. of the return levels should be chosen at any set of points other than those used in working from A to B. Thus a new set of determinations is made which should "check" closely, within about 0.005 of a foot of the first determination.

(74) GENERAL PROBLEM IN LEVELING B - Profile Leveling

This problem will undertake to get the elevation of a series of points, called stations along a line (either straight or crooked) so as to chart the profile.

Assume a line staked off by tape or chain measures giving stations along the line at regular intervals of 100 ft., 50 ft., or any other desired number of feet and levels taken at these and intermediate stations. A line, as shown below, is the horizontal



L_1 L_2

Figure 19.

projection along which a profile is desired. The line A B C D is laid off in stations 0 (at A); 0+50; 1; 1+20 (at B); 2 (which is also used as a turning point); 2+50; 3 (at C); 3+50; 4; and 4+75 (at D). L_1 and L_2 are positions at which the level is set up, the same being about equidistant from B.M. (a bench mark at known or assumed elevation) and from which rod-readings can conveniently be taken at each full or intermediate station. With level set at L. and carefully leveled up, read the rod at B.M.(+sight); then rod-readings at 0, 0+50, 1, 1+20, 2 will be - sight readings.

PROB. NO. 12 PROFILE OF LAFAYETTE AVENUE

Station	B.S.	H.I.	F.S.	Elev.	B.M. + T.P. Elev.	Remarks
B.M.$_1$	3.00	57.61			54.61	S.B in curb
0+00			2.5	55.1		
1+00			5.1	52.5		
2+00			8.6	49.0		
+65			10.9	46.7		
T.P.$_1$	1.67	47.28	12.00		45.61	Steel spike
3+00			3.4	43.9		
+10			2.5	44.8		
+70			5.8	41.5		
4+00			4.0	43.3		
+25			6.0	41.3		
5+00			3.2	44.1		
T.P.$_2$	10.95	54.78	3.45		43.83	Edge curb 85' S of Sta. 5.
6+00			8.4	46.4		
7+00			3.1	51.7		
T.P.$_3$	11.75	64.79	1.74		53.04	Nail in fence 19' W of Sta. 7.
7+50			12.1	52.7		
8+00			7.3	57.5		
+12			6.5	58.3		Check
+60			3.2	61.6		
9+00			2.1	62.7		
T.P.$_4$	9.63	73.44	0.98		63.81	Top of imbedded stone
10+00			10.0	63.4		
11+00			11.0	62.4		
+50			12.1	61.3		
12+00			9.5	63.9		

Equipment:-

Wye Level #7063
Target Rod #27
100-foot steel tape
2 plumb bobs
2 Steel Turning Points
11 Chaining pins
Hammer

Dec. 3, 1909.
Cloudy & cool.

Hoskins, J. L. }
Martin, P. P. } chain
Johnson, F. P. }
Benton, K. T. }

NOTE:-

B.M.$_1$ is top of stone bound set in curb at S.W. corner Lafayette Ave. & Front St., being the N.E. corner of Gregory's Addition.

Arithmetic Check to T.P.$_4$:

+	−
3.00	12.00
1.67	3.45
10.95	1.74
11.75	0.98
+27.37	18.17
−18.17	
+9.20	
54.61	
63.81	

The instrument is then set up at L_2, carefully leveled as before, and a + sight taken upon 2 as a turning point. After this read the rod held at 2+50, 3, 3+50, 4, 4+50, and 4+75. Record in note-book as follows:

PROFILE OF LONEVILLE ROAD

Sta	B.S	H.I.	F.S.	Elevation		
B.M.	4.752	185.752				B.M. 181.000 ft. above Datum.
0			5.312	180.440		
0+50			6.034	179.718		
1+00			6.432	179.320		
1+20			6.793	179.009		
1+50			5.664	179.868		
2+00	8.455	181.205	6.002	179.750		T.P. on wooden stake driven in grou
2+50			8.760	179.445		marked Sta. 2+00 T.P.
3+00			9.502	178.703		
3+50			7.936	180.269		Rod markings taken to thousandths
4+00			5.482	182.723		of a foot.
4+50			4.873	183.332		Plot profile to hundredths
4+75			4.506	183.699		SCALES { 1 inch = 100 ft. Horizon
	13.207				6.002	(1 " = 4 ft. Vertical
	181.000				4.506	
194	194.207				10.508	
10	10.508		check			
183	183.699					

Figure 20.

To find the elevation of H.I. above datum, add the rod-reading on B.M. to height of B.M. above datum, and record under H.I. in the table. - 181.00 + 4.752 gives 185.752. Subtract the station rod-readings (which are all foresights) from H.I. to find their elevation and record under elevation. This is done for each full station (or intermediate station) until the T.P. is reached. Here we add the back-sight on the T.P. to the elevation of T.P. obtained from the previous fore-sight and thus find the H.I. for the second setting of the instrument. From this point on, the rod-readings are again all fore-sights, which are now subtracted from the new H.I. to find the elevations of the remaining stations.

A careful study of the notes and the accompanying profile chart will give a clear notion of the meanings of the various quantities and of the graphical means of illustrating them.

The scales used in charting are usually chosen with a view to the purpose of the work:- here to show the rise and fall of a road grade line. The horizontal scale is rather small but for the purpose amply large to give necessary details; while the vertical scale is comparatively large, or exaggerated to show elevations in greater relief. This is customary although the relative scale values are usually of much lower ratio.

Other leveling problems will be taken up in Assignments XIII and XIV.

PROBLEMS

1. The following set of level notes taken for finding the difference in elevation between B.M.$_1$ and B.M.$_2$ give the tabulated data. Find the difference of elevation.

Sta.	B.S.	H.I.	F.S.	Diff.
B.M.$_1$	8.78			
T.P.$_1$	6.81		4.83	
T.P.$_2$	2.74		7.68	
T.P.$_3$	11.41		5.82	
T.P.$_4$	0.75		9.74	
B.M.$_2$			6.81	

2. From the following tabulated field notes find the elevations of all stations on the line; sketch the profile.

Sta.	B.S.	H.I.	F.S.	Diff. El	Elev.
B.M.$_7$	6.23				125.75
0 + 00			6.03		
0 + 25			6.45		
+ 75			7.14		
1 + 00			6.10		
+ 50			5.16		
2 + 00			6.01		
+ 25			6.50		
+ 50			6.83		
+ 75			7.04		
T.P.$_1$	7.34		5.63		
3 + 50			6.45		
+ 75			7.16		
4 + 00			8.17		
+ 25			8.49		
+ 50			7.83		
+ 75			7.20		
5 + 00			6.85		

References:

Tracy, Chap. XXI

Raymond, pp. 50 - 60

Johnson, pp. 71 - 78

Breed & Hosmer, Vol. I, pp. 232 - 247

Assignment 9

THE TRANSIT AND ITS USES

FOREWORD:

The purpose of this assignment is to describe at length the transit, and to acquaint the student as far as possible with the wide range of its uses in surveying. Problems of a general character requiring the transit in their solution will be dealt with in a subsequent assignment.

(75) THE TRANSIT

The transit, a truly universal instrument for surveying and engineering work, is for the work of surveying in all its branches by far the best and most adaptable. In a complete engineer's transit, such as that illustrated in Plate III, are combined the compass, the level, the theodolite, and the tachymeter. It is used to measure angles in both a horizontal and a vertical plane; to determine heights and distances, magnetic or true bearings, and angles in azimuth and altitude. In its most refined forms, it is capable of the nicest or many desired determinations of such quantities as are named above. It enables the engineer to accomplish his work with great precision and satisfaction. With the aid of suitable attachments the transit is capable of solving many of the intricate problems confronting the engineer in the realm of astronomy.

PLANE SURVEYING

COURSE X–1A

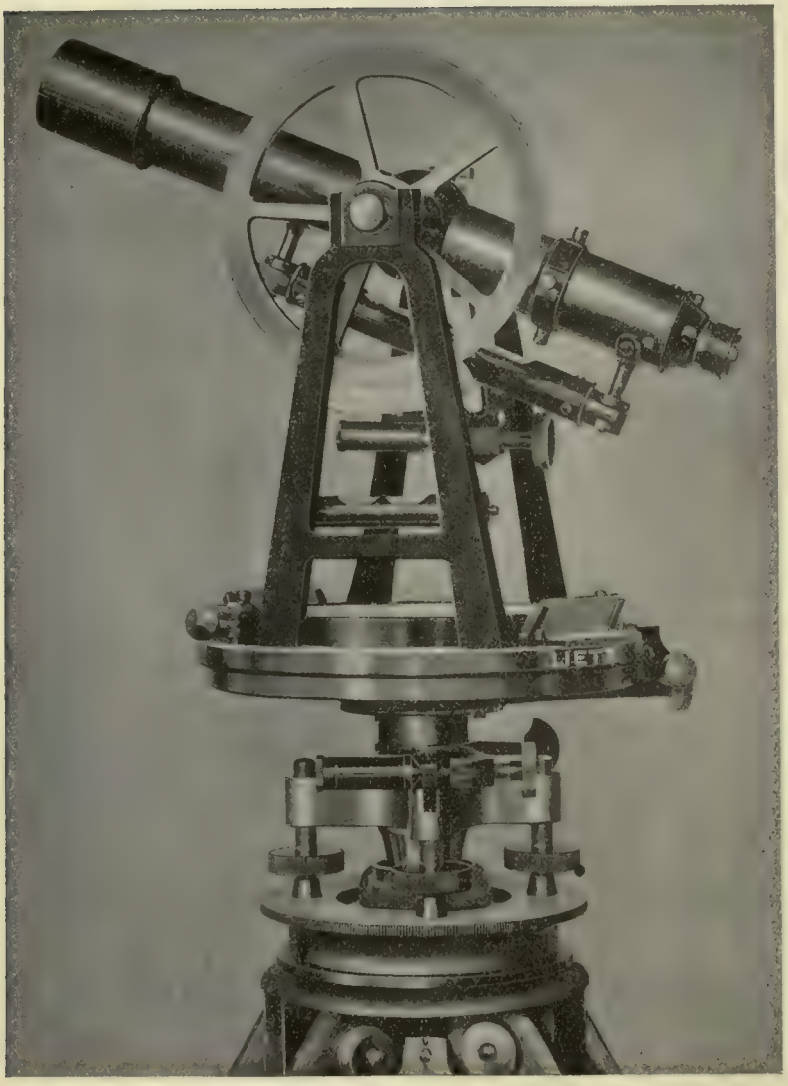


PLATE III

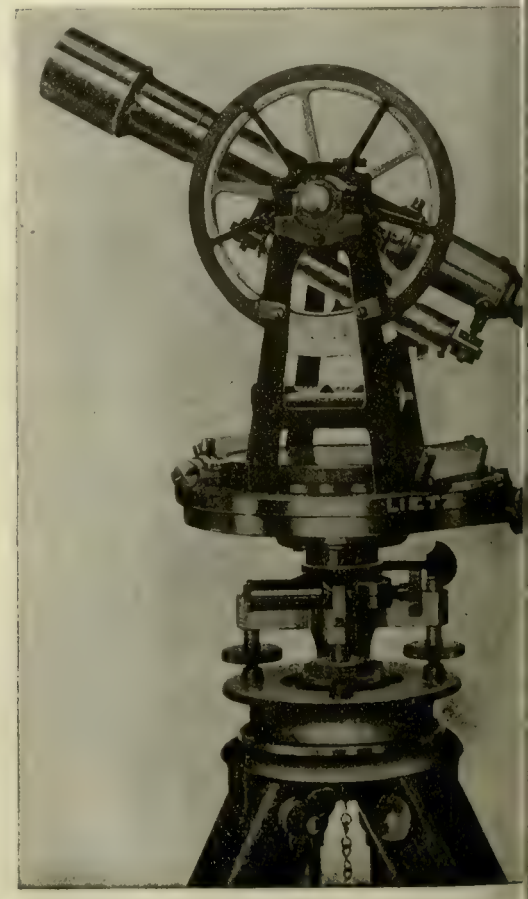
COMPLETE ENGINEERS' TRANSIT

Fitted with 5-inch vertical circle, provided with a double vernier, reading to minutes.

PLATE IV

MOUNTAIN AND MINING TRANSIT

Fitted with 4-inch full vertical circle graduat solid silver, vernier reading to minutes, alum protection guard, and full extension tripod.



(76) TRANSIT DESCRIBED

A transit comprises primarily a telescope similar in construction to that described under the LEVEL. The telescope is mounted upon a plate bearing two standards in which it turns completely around on trunnions in a vertical plane and the plate, standards, and consequently the telescope, may also be moved about a vertical axis in a horizontal plane. This part, then, called the alidade, has two degrees of freedom of motion, i.e. it may be turned in azimuth and in altitude.

(77) THE PLATES

The plates of the transit are really three, but generally only two are spoken of. The plate to which the standards are attached, the alidade plate, rests upon a second plate with which the first is concentric. A vertical axis may move about through their centers together or independently on carefully constructed spindles. These spindles fit one within the other and both into a suitable socket bored into a third plate which forms the base of the instrument. The third plate, sometimes called the foot-plate, is securely attached to the supporting tripod and forms virtually the tripod-head.

The alidade-plate turns on its spindle over the lower plate; the lower plate turns about its spindle over the foot-plate. The foot-plate is securely fixed, usually upon a tripod that stands on the earth. Again the alidade-plate carries one, and generally two circular-arc verniers. The lower plate is graduated in degrees and fractions, by which angles in the horizontal plane may be

measured. These verniers which are needed to measure small fractions of angles will be fully described later on.

(78) CLAMP-SCREWS, TANGENT-SCREWS

As stated above the alidade-plate and the graduated-plate, the outer graduated rim of which is called the limb, may be moved together or separately in azimuth. In order that this may be accomplished, each plate has a clamp-screw by which the upper may be secured to the lower or the lower to the foot-plate. When in this clamped position it is desirable to rotate either plate through a small angle, for careful and nice adjustments, each plate is furnished with a tangent-screw to accomplish this object. They are called tangent-screws, because they act along a tangent to what may be assumed to be the plate's circumference. The location and the form of each of the above screws are such as to enable the instrument man to find a particular one at pleasure. An examination of a transit or of the detailed illustration, Plate V, will reveal them to you.

(79) LEVELING-SCREWS

Attached to the foot-plate, in which the spindles are fitted and in which they turn, are four (or in some instruments only three) lugs extending radially, into which fit smooth, well-cut screws, having large milled heads, one in each lug. These screws, called foot-screws, rest upon the fixed tripod head and permit the spindles to be shifted until they are brought into a vertical position. Since the two plates are fixed at right angles, each to its own

spindle, by adjusting the spindles until they are vertical, the plates are made horizontal at the same time.

(80) CLAMP AND TANGENT SCREW ON TELESCOPE

Attached to the right hand standard are the clamp to lock the horizontal axis at the trunnion, and the tangent screw to give it gradual motion through a small angle in the vertical plane of the telescope. The three tangent screws now described bear against opposing springs that hold the moving part firmly, except when pushed forward or withdrawn backward by the screw.

(81) LEVELS UPON PLATE

The alidade-plate has two small spirit-levels attached at right angles to each other by adjustable screws, which enable the levels to be brought into parallelism with the plate, i.e. at right angles to the vertical axis through the spindles. In some transits, one of these levels is attached to the left hand standard that supports the telescope, but its office is the same as if it werelying upon the plate.

(82) COMPASS AND MAGNETIC NEEDLE

Commonly a transit has a compass plate and a magnetic needle fixed to the alidade-plate, by which magnetic bearings may be taken, and magnetic declinations may be determined. The magnetic needle is of especial value in checking angles measured upon the graduated or lower plate of the instrument.

(83) LEVEL ON TELESCOPE

A spirit-level, several inches in length, and usually of

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COURSE X–1A

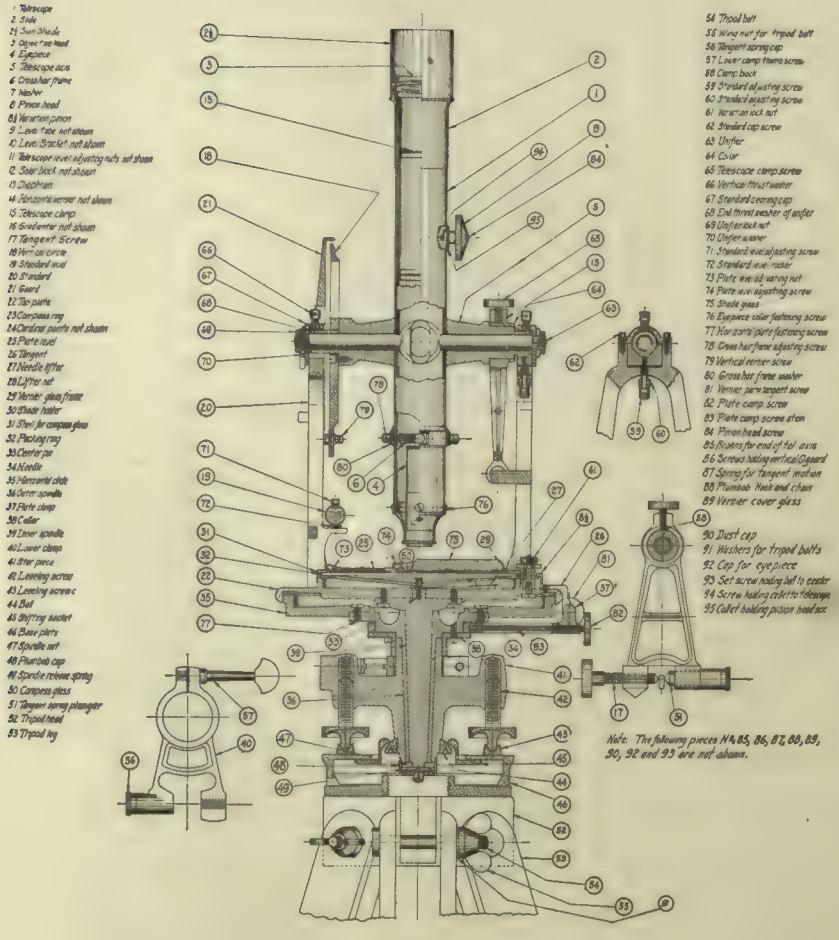

PLATE V

Cross-section of a Transit.

CORRECTION

No. 29 should read "Shade Holder" instead of "Vernier Glass Frame."

No. 30 should read "Vernier Glass Frame" instead of "Shade Holder."

great sensitiveness, is connected parallel to the telescope by means of short columns. By a use of this level the line of sight may be made horizontal as in the engineer's leveling instruments already described. This enables the engineer to fix the horizontal plane for the usual purposes of leveling, or for securing angles of elevation or depression, etc.

(84) VERTICAL CIRCLE OR VERTICAL ARC

Attached to the horizontal axle of the telescope (with complete transit) is a graduated circle or a graduated arc, with accompanying verniers, for measuring vertical angles (angles of elevation or of depression).

(85) OTHER ACCESSORIES

A variation arc is sometimes connected with the compass-plate. It is often of great convenience in setting off the declination.

A gradienter screw is in some transits substituted for the usual tangent-screw to the horizontal axis. By this small angles in elevation (or depression) may be set or determined independent of the vertical arc. It is a screw of very refined make, the thread of which and the divisions on its head being so graduated as to secure an elevation of one foot at a distance of 100 feet or 200 feet for one full turn of the screw.

(86) THE GRADUATION OF THE LOWER PLATE

The lower plate is graduated in degrees and fractions of degrees, usually to halves or thirds; i.e., the smallest division

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COURSE X–1A

METHODS OF GRADUATING SURVEYING INSTRUMENTS AND POPULAR STYLES OF VERNIERS FURNISHED

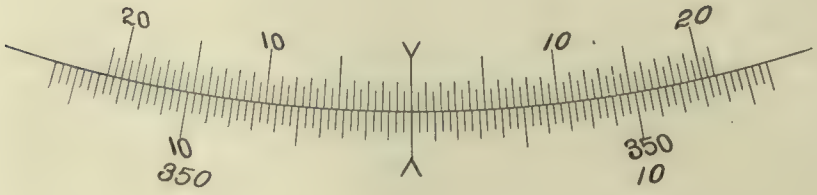

FIG. 21

Double vernier reading to 30″. Circle graduated to 20′.

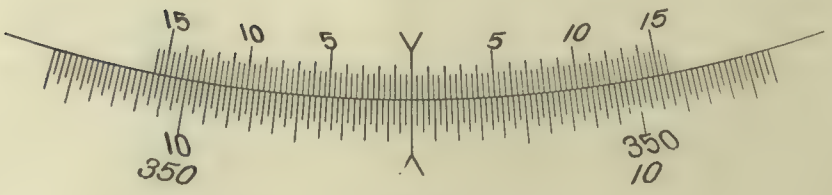

FIG. 22

Double vernier reading to 20″. Circle graduated to 15′.

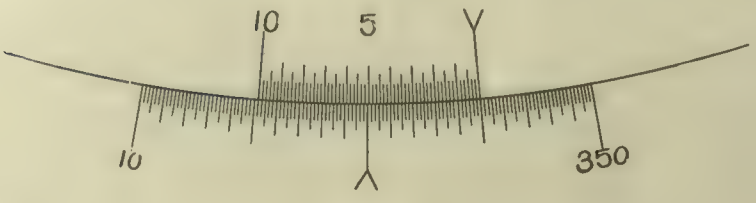

FIG. 23

Single vernier reading to 10″. Circle graduated to 10′ with one row of figures.

on the graduated limb is 30 minutes or 20 minutes of arc. In some instruments the division is carried to 10 minutes of arc but this is rather exceptional, being too small for ease in reading, and of no especial advantage.

The divisions are grouped in spaces of 5 and 10 degrees marked on the scale by lines somewhat longer than the usual division lines and numbered at the 10, 20, 30, etc., marks. The zero of the scale is indicated by a 0 or ∧. The numbering is generally in both directions, preferably through 360°; one set of numbers being nearer to the inner part of the graduated ring and reading clockwise; the outer set reading, therefore, counter-clockwise. For convenience in reading, the characters are slanted in the direction of numbering. Some makers color the inner or clockwise set in black, the outer or counter-clockwise in red. These and other devices are intended to assist in reading in the right direction and to prevent confusion of scales.

(87) VERNIERS

The vernier as adapted to reading linear scales (see Assignment 8) has already been explained. Now we turn to the circular vernier.

On the level-rod, verniers of two kinds are used, the direct-reading vernier and the retrograde vernier. As the retrograde vernier is rarely used on transits, the direct type only need be discussed here.

One form of vernier is constructed by taking 29 degrees

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CORRESPONDENCE COURSES IN ENGINEERING SUBJECTS

PLANE SURVEYING

COURSE X–1A

METHODS OF GRADUATING SURVEYING INSTRUMENTS AND POPULAR STYLES OF VERNIERS FURNISHED

FIG. 24

Single vernier reading to 20″. Circle graduated to 20′ with two rows of figures.

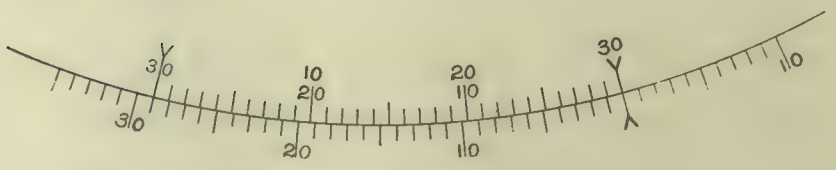


FIG. 25

Vernier reading to 2′. Circle graduated to single degree.

FIG. 26

Double vernier reading to single minutes. Circle graduated to 30′.

(when the smallest division on the limb is the whole degree) and subdividing it into 30 equal parts for the vernier scale. The value then of one division on the vernier will differ by 1/30 of 1 degree, or 2', from a single division on the limb; this value is called the "least count" of the vernier. Another commonly used vernier is formed by subdividing 29 half-degrees (the smallest division on the limb being a 30' space) into 30 equal parts on the vernier scale; here we have $\frac{30'}{29 + 1} = 1'$ as the least count. A vernier constructed upon a scale of 20 minute divisions in which 39 of these divisions are divided into 40 parts on the vernier would give a least count of 1/2 minute or 30"; thus if 20' is the smallest limb division and 39 the vernier number, then $\frac{20'}{39 + 1}$ will equal $\frac{20'}{40} = 1/2' = 30"$. The following rule may now be given:

RULE. To find the least count of a vernier:

Divide the value of the smallest division on the limb by the number of such divisions plus one required in forming the vernier scale. In general, To find the least count of a vernier:

Let l.c. = least count, d = the value in circular units of the smallest division on the limb, n the number of these smallest divisions that make n+1 spaces on the vernier-scale; then

$$l.c. = \frac{d}{n + 1}$$

In this formula the resulting unit value will be that unit in which d is expressed. This of course may then be reduced to any required unit.

Verniers are of two types, single and double. The double type comprises two single verniers extending in opposite directions from the initial point, or index, the numbering also extending in opposite ways. Thus, with this double vernier, when the alidade plate has moved over the graduated limb clockwise, the left-hand vernier is observed; but when in counter-clockwise direction, the right-hand vernier is used, always reading (for a direct vernier) in the direction of motion. The vernier reading is then appended or added to the reading on the limb.

For economy of space another plan of laying out the vernier scale is sometimes used. In this style one half of the vernier is to the right, the other to the left of the index mark. This form is called the "folding vernier" or "split vernier" and is shown in Figure 26A. Following what has already been said respecting

Folding or "Split" Vernier

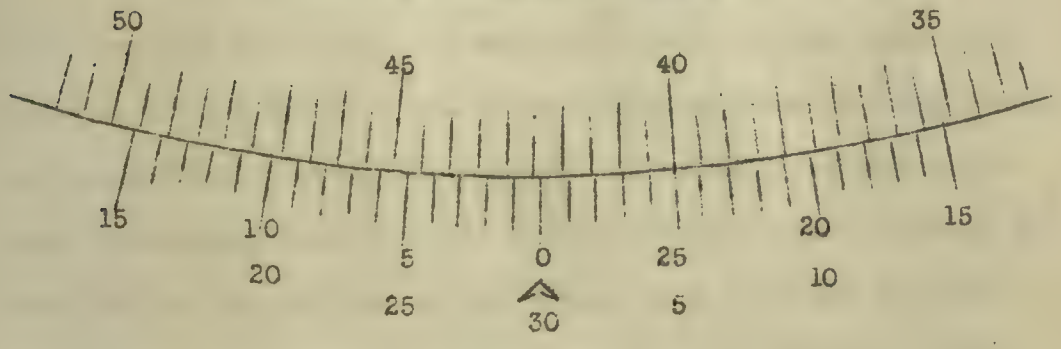
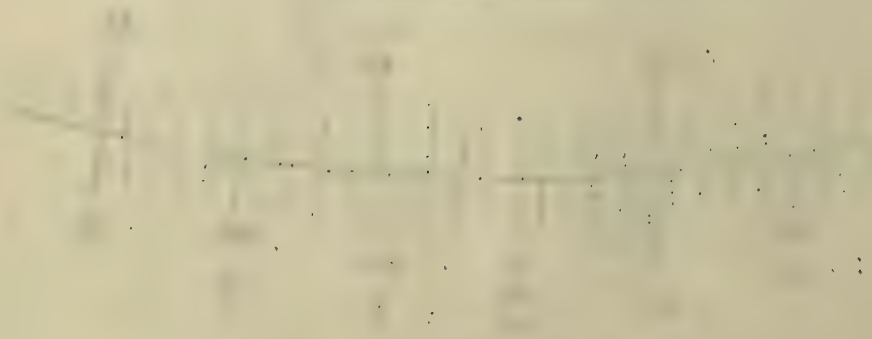


Figure 26a.

verniers in general, no extended description of the folding type is needed. The method of reading the folding vernier is as follows:

Read the limb to the last full division passed over by the vernier's index; then determine the vernier reading by counting



along the vernier in the direction of motion of the alidade-plate to the line of coincidence as usual. If the coincidence does not occur over the first half of the vernier scale, then continue the count from the opposite end of the vernier toward the index, to the line of coincidence. Add the vernier reading to the reading on the limb.

Here the smallest division on the limb is 30'; 29 smallest intervals are divided into 30 parts on the vernier; hence the least count of the vernier is one minute. In the illustration the movement of the alidade plate taken clockwise has brought the vernier index past 42°; reading the vernier forward to the left, we find no coincidence to the left half and, therefore, returning to the extreme right and again counting forward to the left the coincidence is found at 25. This appended to the limb reading gives 42°25' as the full, correct reading.

Much has been said and several illustrations have been given covering the subject of verniers. This has been done in an endeavor to make clear to the student this important and well-nigh indispensable adjunct of the engineer's transit. The student is urged to acquaint himself fully with verniers of every variety, in order that he may be able to read angles with ease and dispatch. Also in using any transit or other instrument on which a vernier must be read, be careful to determine its "least count" first of all.

PLANE SURVEYING

COURSE X–1A

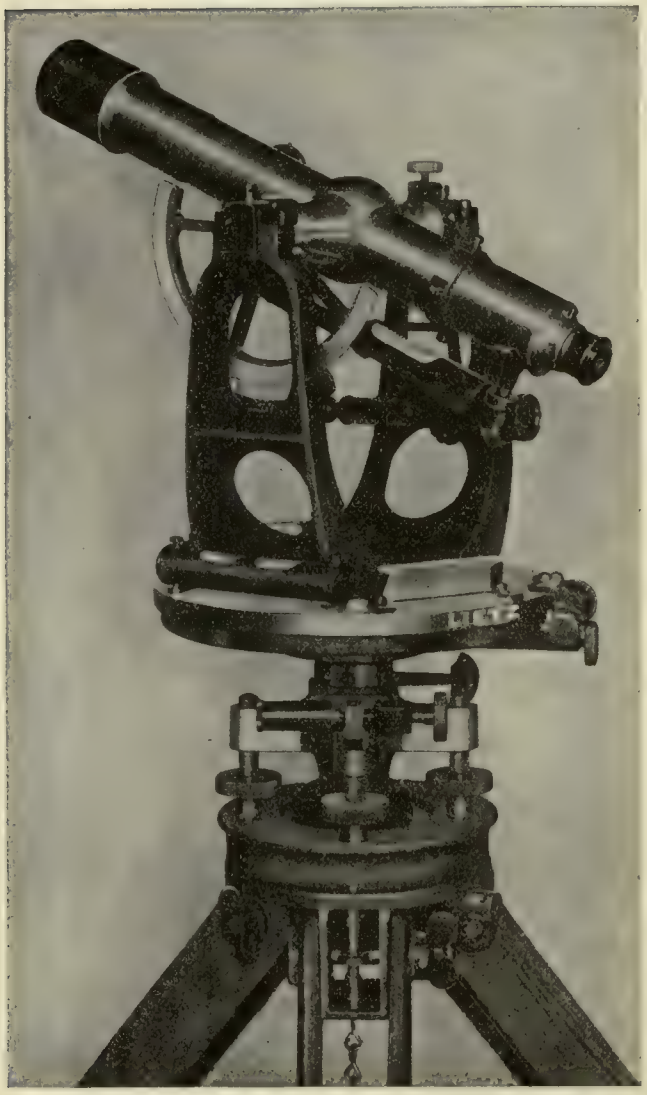


PLATE VI

TRANSIT THEODOLITE

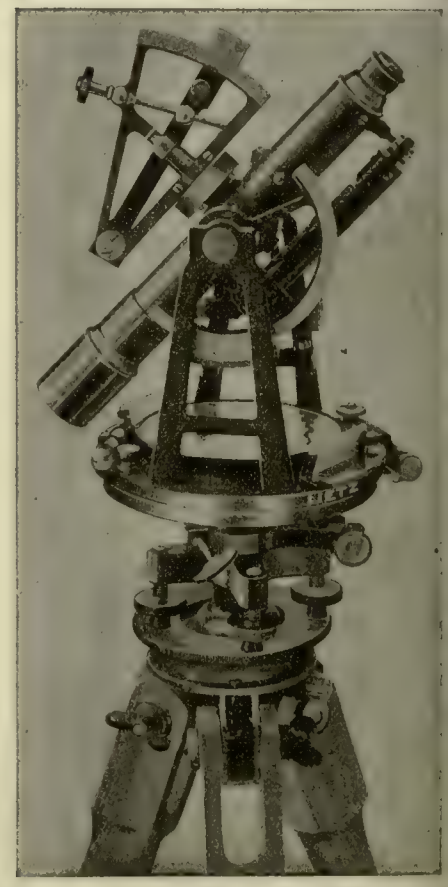


PLATE VII

MOUNTAIN AND MINING TRANSIT
FITTED WITH
BURT SOLAR ATTACHMENT

(38) THE VARIOUS USES OF THE TRANSIT

A. Ranging-out Lines.

The fact that the transit telescope can be rotated in both a vertical and a horizontal plane enables us to make use of it for determining direction and for prolonging lines at any angle desired.

For purposes of surveying, lines must be continued or prolonged in certain desired positions upon the earth's surface, as in setting out a line of division between properties, a boundary line or fence line. Such a line must often be carried from point to point either in a straight or more often in a broken or curved line. Ranging out lines is also practiced in road, railroad, canal, ditch, and other construction, and in most problems in land surveying. (See especially Survey of the Public Lands, Assignment XIX.)

Through homogeneous media light is propagated in straight lines; through any such media, therefore, light emanating from any distant source is brought to the eye in a path that does not deviate from a straight line unless its course be interfered with or deflected by intervening agency causing reflection or refraction. This principle is the fundamental one in all uses of the telescope in surveying and in astronomical measurement. As mentioned in the description of the telescope on the engineer's level, the line of collimation may be prolonged indefinitely into "the line of sight". Observe that when the transit telescope is directed toward a distant object, light from that object enters the large lens and is

focused at the intersection of the cross-hairs. In fact, a line joining the object viewed and the mid-point of the two cross-hairs is a straight line traversed by a ray of light. We are certain then that we can retrace this line of sight as a straight line, unless it is bent from the straight course, by something that deflects it - a reflector placed in its path, or media of rarer or denser kind which changes its direction. But the purposes and the practice of surveying are such as to enable us to eliminate these interferences or to rectify them with directness and fidelity to the principle of "sighting".

The line of sight in the transit telescope may be directed up or down by its rotation upon the horizontal axis or to right or left by turning the plate and the telescope supports about the vertical axis. Therefore:

To prolong a straight line

Set up the transit* at one extremity of the line; place a marker at the other end; fix the cross-hairs upon the marker (in common parlance "bisect the point"). Now by shifting the telescope upward about the horizontal axis (i.e. by turning it upon its trunnions) extend the line of sight beyond the farther extremity of the given line and place markers to fix it upon the ground. Markers, as pins and stakes, or rods are commonly those described under "measuring lines with chain or tape".

* To "set up a transit" means to place it so that the vertical axis points to the zenith through a point of occupancy on the earth. In that case the plumb-line attached to the transit axis will be directly in line with the given point, when the plate-bubbles are at center. (It is assumed that the instrument is in adjustment as far as the vertical axis and plate levels are concerned.) See Assignment XIV on adjustments.

If now it is desired to prolong the line in the opposite direction, clamp the plates, adjust the line of sight to bisect the most distant point of the line, in its original length or prolonged, and invert the telescope on the trunnions so that the line of sight may be directed in the opposite direction; place markers (pins or stakes) as before.

Instead of inverting the telescope (called plunging) it may be reversed in direction by turning the alidade-plate through a horizontal angle of 180°. This assumes that there is no eccentricity either of the alidade-plate or of the vernier indexes by which the angle of 180° is determined, in other words that these parts are in adjustment. If the plate adjustments are not true, the line will not be straight but bent at the point over which the instrument is set.

To eliminate such an error use a principle called double sighting.

(89) DOUBLE SIGHTING

Double sighting consists in sighting upon one extremity, B, of a line when the transit occupies the other extremity A; this brings the line of sight in a vertical plane through the line joining the two points. Turn the alidade-plate through 180°; sight the telescope upon a required point C, in the reverse direction; now reverse through 180° bisecting the first point B; plunge the telescope (i.e. invert it) and if the point C is again bisected the line AB, and its prolongation AC, form a straight line; if not, choose a point midway

between C and the point C' next it; this mid-point will lie in the prolongation of AB, as required. In the above method by double-sighting it is assumed that the vertical axis is truly vertical and that the horizontal axis is truly at right angles to it. (Sec. Adjustment of the Transit, Assignment XIV.)

(90) B, MEASURING HORIZONTAL ANGLES

The transit is used to measure angles in the horizontal plane. These angles are the bearings of lines, deflection angles for change of direction, azimuths from any assumed point of direction, or angles included between any two lines, and hence directly the interior angles of any concave figure, as those of a triangle or other polygon in a horizontal plane.

(a) To measure angles of bearing:

Set up the transit over one extremity of the line; set the vernier-index (that of the A vernier) on zero of the graduated plate. With the alidade plate clamped in this position, and the lower motion screw unclamped direct the line of sight in line of the assumed meridian (magnetic, true, or any other chosen meridian). Now clamp the lower motion adjusting by means of the lower tangent screw. Unclamp the alidade-plate and sight on a rod or marker held on the line whose bearing is required and read the angle moved over by the vernier index. This will be the angle of bearing. (Compare this method with that explained in Compass Surveying, Assignment VI.)

In reading the angle always note carefully the direction in which the vernier index moves and observe the numbering on the plate

for this movement - the inner set of figures on the limb, if the movement is clockwise; the outer set of figures, if the movement is counter-clockwise. The bearing would be recorded as N 23°12'E, or N23°12'W, or as S23°12'E (or W) as the case may be. Bearings must always be so designated. (See Compass Surveying.)

(b) To measure a deflection angle;

Set up the transit over one extremity of a line A. With A vernier set at zero of the limb and the alidade-plate clamped, plunge the telescope (i.e. invert from its normal position, which means with the eye-end near the A vernier). With the telescope in this inverted position sight upon a marker (a rod, pin, or stake) held at the other extremity of the line B, the lower motion being used for this purpose. Clamp the lower motion and by means of the tangent-screw of the lower motion bisect the marker. Turn the telescope back to normal. The line of sight is now in the prolongation of the line AB. Unclamp the alidade-plate, sight in the direction of the deflected line AC; clamp the upper plate and bring the line of sight exactly upon the point selected in AC; i.e. bisect it. Read the angle and record its deflection as right or left in accordance with the movement of the alidade; thus, Defleetion 18°27' R (right), or Defl. 18°27' L (left), as the case may be. It is always essential that the designation right (R) or left (L) ge given in registering deflection angles.

The further uses of the transit will be continued in the next assignment.

QUESTIONS

1. Name some advantages in the use of the transit as compared to the compass.

2. A vernier is constructed by dividing 49 thirty minute divisions into 50 equal parts; what is the least-count of this vernier? (a) In degrees? (b) in minutes? (c) in seconds?

3. By a figure neatly and carefully drawn show what you understand by double sighting as given in paragraph 89 in this assignment.

4. The deflection angles of a survey of closed traverse are as follows: 42°16', 111°17', 118°45', 87°40'. (a) What is the angular error of closure? (b) Distribute the error to the least two angles and give the value of the interior angles of the field.

References:

Tracy, Chap. XI.
Raymond, pp. 100 - 110.
Johnson, pp. 93 - 100.
Breed & Hosmer, Vol. I, Chap. III.

UNIVERSITY OF CALIFORNIA EXTENSION DIVISION

Correspondence Courses

Surveying-1A Elements of Surveying

Assignment 10

The Transit and its Uses

91) RANGING LINES AND MEASURING ANGLES

The varied uses of the transit in surveying and engineering rest upon two principal functions, the sighting of straight lines and the measurement of vertical and horizontal angles. This assignment therefore will deal with a few of the fundamental uses of the transit in ranging lines and measuring angles. A knowledge of these fundamentals will enable you to employ the transit for many other uses.

A line is said to be "ranged out" when a line segment or any two points, or a known point and the line-direction are given. If a line segment is given and it is required to extend this line in either or both directions, the method is usually the same as that explained in the preceding assignment. Set up the transit over any point in the segment; sight on a marker placed on any other point in the segment; clamp both plates, and fix other points in the line by sighting through the telescope. If it is required to extend the line both forward and backward, after sighting points, say in the forward direction, plunge the telescope and proceed in like manner in the opposite direction.

In case two points are given conceive these as the termini of a line segment and proceed in the same manner.

Given a point in a line and the angle of direction, i.e. its bearing or direction with respect to some other line of fixed position. Let the point be A and the angle α. Set up the transit at A; with vernier set at 0, turn on the lower motion until the line of sight is on the line of known direction by sighting at any point in the known line other than A; clamp the lower motion; unclamp the upper motion (the alidade plate); turn off the angle α (right or left as the case may be), making the final more delicate adjustment with the tangent screw. The telescope is now in the line required; that is, its direction and one point (A) are determined. Place a marker to fix upon the ground the new line thus determined.

In ranging out railroads, wagon roads, canals, ditches, and in fact almost all lines, the lines are broken and are formed of many segments of greater or smaller size. When such a series of lines is run, the road is said to deflect (right or left) at certain designated points in the general route of the road. In running these segments, therefore, it is necessary to set up the transit (or occupy) each point where a change of direction or deflection takes place. The transit telescope is then brought into line with the last segment, the upper motion being clamped, and the lower motion left free to turn. The deflection angle is then turned off with the upper plate unclamped. In this case the final and nice adjustment should be made by means of the vernier and tangent-screw. The new segment, or such portion as can conveniently be

sighted, is then established and the work can proceed to the next station or point of change of direction.

To bring the telescope in line with any previous segment, after setting up over a point at which deflection takes place, it is the best practice to proceed as follows: Clamp the alidade plate, fixing the vernier at zero; unclamp the lower plate, plunge the telescope, and backsight on a point in the segment just established; now clamp the lower plate and return the telescope to normal position (i.e. re-plunge it). It is now pointing in the direction of the last segment prolonged through the point occupied by the transit. Now turn off the deflection angle on the graduated plate and proceed as in other cases.

This method requires that the transit be in perfect adjustment and especially that the horizontal axis of the telescope be truly horizontal and at right angles to the line of collimation of the instrument. (See chapter XIV on Adjustment of the Transit.) To avoid error, in case these conditions do not obtain, the following simple method may be substituted for the latter favorite method; With alidade plate clamped (vernier at zero) and telescope normal, sight in a backward direction to the previous segment; then add 180° to the deflection angle and turn off this sum on the graduated plate with lower motion clamped. The telescope will then be in line of the new segment as before. (Caution. Be sure that the angle turned off is the sum of 180° plus the deflection angle and that the limb is read in the proper direction.)

PROB. NO. 7. TRAVERSE OF FIELD WITH TRANSIT AND TAPE - DEFLECTION METHOD

Station	Deflection Angles Left	Deflection Angles Right	Computed Bearings	Obs. Mag. Bear. F.S.	Obs. Mag. Bear. B.S.	Measured Distances Ft.	Corrected Distances Ft.
A		86°31'					
			S 0°00'E	S 32°45'E	N 33°05'W	334.97	335.00
B	10°13'						
			S10°13'E	S 43°25'E	N 43°20'W	464.93	464.98
C		124°53'					
			N65°20'W	S81°35'W	N81°35'E	483.67	483.72
D		76°03'					
			N10°43E	N22°20'W	S22°25'E	616.47	616.53
E		82°46'					
			S86°31'E	S60°10'E	N60°35'W	242.82	242.84
A							
	10°13'	370°13'					
		-10°13'					
	(Check)	360°00'					

CALCULATION OF BEARINGS.

AB	S 0°00'E
B	10°13'L
BC	S10°13'E
C	124°53'R
CD	N65°20'W
D	76°03'R
DE	N10°43'E
E	82°45'R
EA	S86°31'E
A	86°31'R (Check)

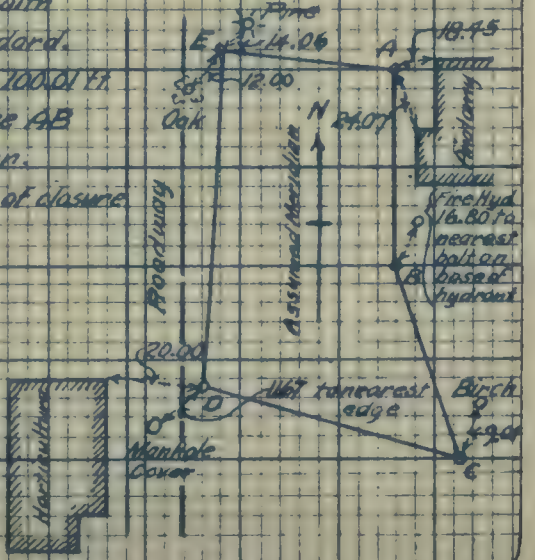
Equipment:
K & E Transit #10753
100-foot steel tape #74.
2 Plumb-bobs.
2 Line rods.
Chaining pins, reading glass.

Sept. 24, 1907.
Clear and cool.

Hart, E. P. } Chain-
Wallace, K. } men
Warren, W. W. } Rod

NOTE:
Measured each deflection angle by inverting telescope, setting "A" vernier on 0°00', sighting on previous station, plunging telescope and sighting on forward station. Checked angle by repeating measurement with the telescope normal for backsight, the "A" vernier being clamped at foresight reading for backsight. If the doubled angle agreed with first value within 01', the angle was considered correct. Recorded magnetic bearings.

Compared tape with University Standard.
Length of tape = 100.01 Ft.
Assumed course AB as true meridian.
Allowable error of closure = 0°01' of arc.



(92) MEASURING ANGLES WITH TRANSIT

In the assignment on The Compass and Its Uses full instruction was given on measuring and setting off angles by means of that instrument. The degree of accuracy secured with the transit in such work is much greater; and the ease of setting off and of sighting by aid of the telescope renders the transit the best instrument available for such work.

The best forms of surveyor's compass are capable in good hands of measuring to the nearest five minutes of arc with a probable error of perhaps one to two minutes, since to read the compass limb to 5 minutes requires that estimation be resorted to in the last analysis. The transit, however, in its best forms, is capable of reading directly to the nearest ten seconds of arc, and by methods in use by the skilled engineer a really good transit may give reliable determinations of angles to the fraction of a second with a probable error of perhaps one tenth to two tenths of a second.

It is only in the most refined measurements, however, that any such degree of accuracy is required, most work being sufficiently close when readings of angles are made to the minute of arc. But this is easily attained by means of the transit, most instruments reading directly to the nearest minute on their verniers, and some to 10 seconds. By the method of repetition (explained further on in this assignment) the refinement may, if desired, be carried to within a few seconds.

As an exercise in transit use in reading angles, set up the instrument over some point, A, with alidade plate clamped and lower unclamped, sight upon some point B, clamp the lower motion, and bisect B by means of tangent screw. Read the limb of the graduated plate to the nearest division as given by the vernier (you should already have determined the least count of the vernier). Unclamp the graduated plate; sight upon a third point, C; clamp the graduated plate and bisect this point by means of its tangent screw; and read the limb as before. The difference between the two readings will be the angle BAC. This measurement should now be checked.

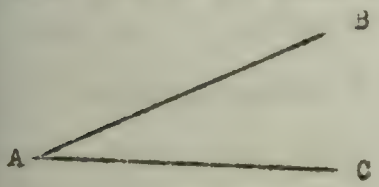
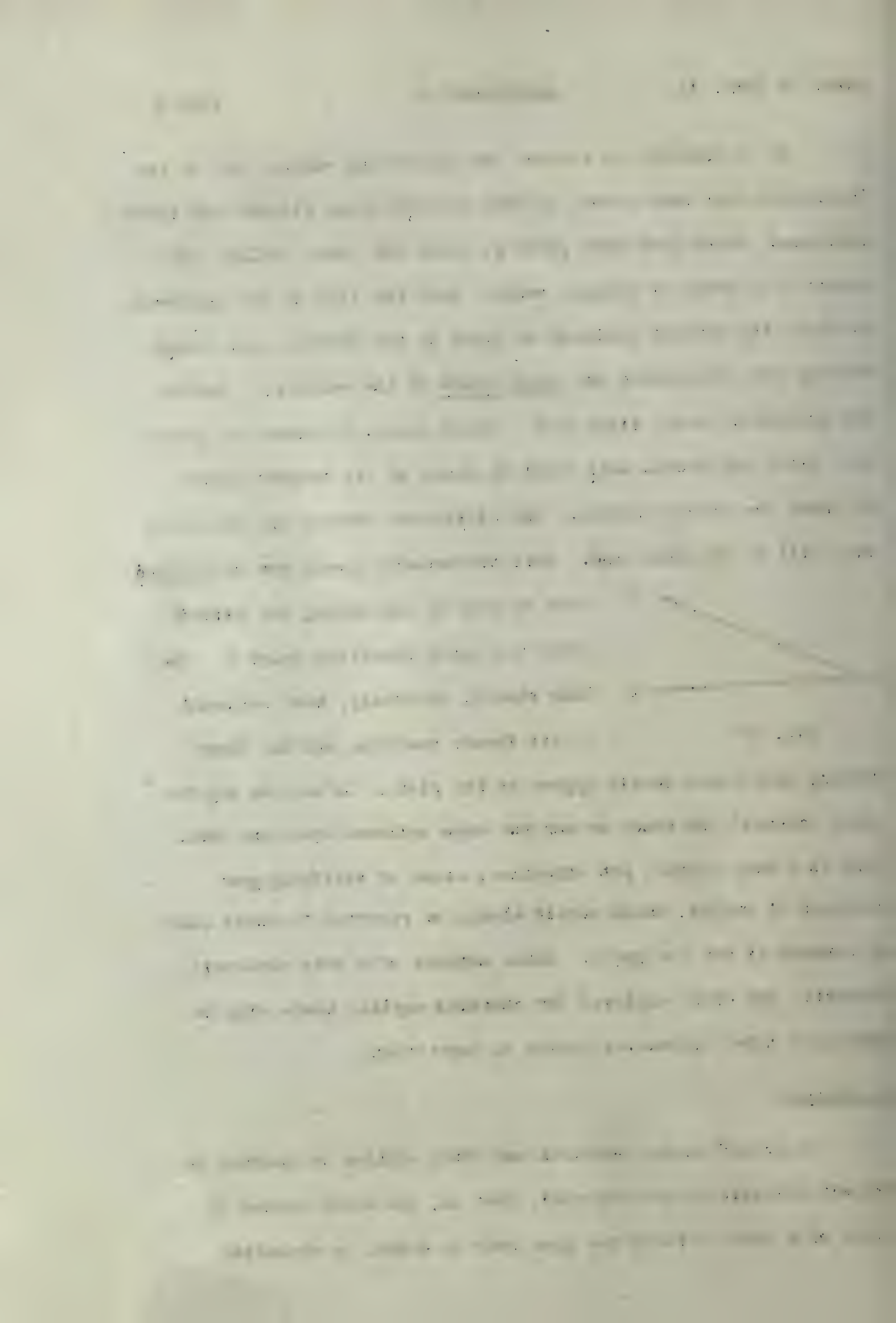


Fig. 27

This is done by unclamping the alidade plate and again bisecting point B. The limb should, obviously, have returned to its former position and the first reading made above should appear on the plate. If so, we say the angle "checks" and hence we can put more reliance upon our work. This is a very simple, yet effective, means of verifying your readings of angles, which should always be resorted to where time or expense is not too great. Other methods of a more elaborate character are often employed for checking angles; these will be explained under "Measuring Angles by Repetition".

(3) TRAVERSING

Lines and angles measured one after another by compass or transit are said to be traversed; that is, the whole series of lines of a road or field are gone over in order, by measuring



lengths with chain or tape (or by stadia) and taking angles by instrument. We have already explained traversing by means of compass and chain in Compass Surveying. Traversing by means of transit and tape will be explained here. And while in case of compass work the chain gave sufficient accuracy of line measurement, a more exact or refined set of determinations is secured by use of transit and tape. A steel tape with graduations in feet, tenths, and hundredths of feet is most suitable for use with a transit reading angles to minutes and even seconds.

Traverses are of two classes: open, such as lines of wagon road, street, railroad, canal, etc.; and closed, such as fields of various polygonal forms. In the latter the traverse is said to close, if, after having gone completely around, the last point as determined by measured sides and angles, falls exactly at the point of beginning. As this is seldom the case, there is then a greater or less degree of "error of closure" caused either by measurement of lines or measurement of angles, or by both, the presumption being that errors arise from the two sources.

The detection and adjustment of errors due to both causes are not so easily made in open traverses as in those which by their very nature close.

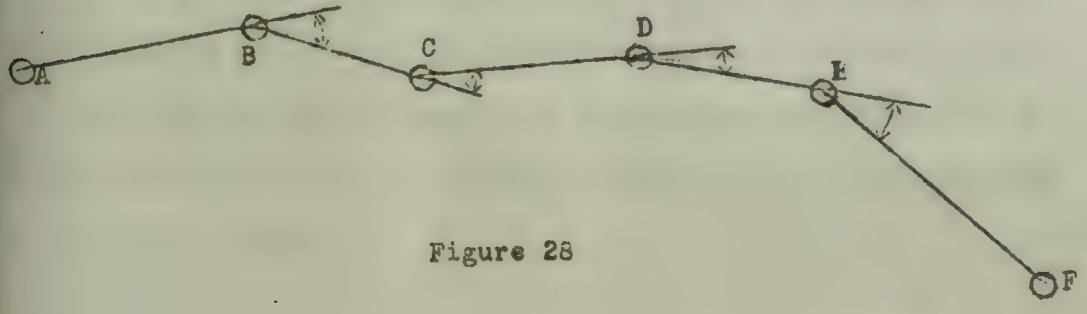
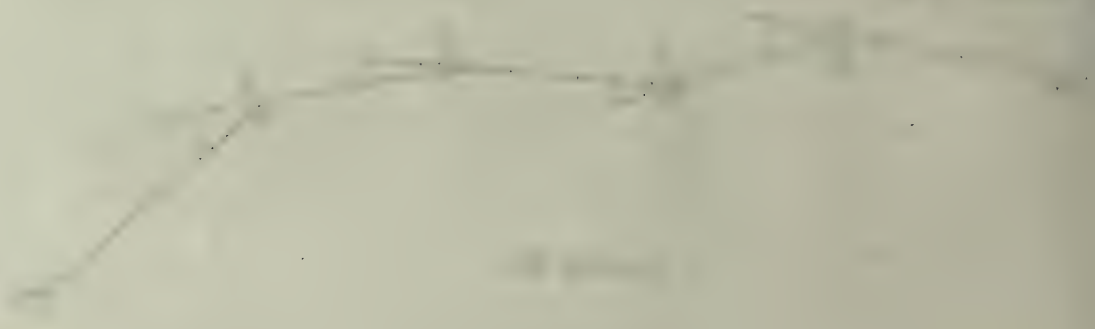


Figure 28



A line of road starting at A and ending at F with deflections at B, C, D, E (Fig. 28) will afford no such check as that of closure to be found in case of a field of the form (or any other form) shown in the accompanying figure (Fig. 29). Here is shown what may graphically represent an error of closure M'M which is a line in magnitude and direction sufficient to close the polygon. Evidently, if the sum of the deflection (exterior) angles be greater or less than 360°, error is manifest, but may be due either to line measure or to angle measure, or to both. Such error is usually, in practice, distributed by proportion to the sides and angles so as to satisfy conditions and cause the figure to close in fact as well as in theory. The manner of dealing with this phase of the problem will be treated in Assignment XVI on Land Surveying.

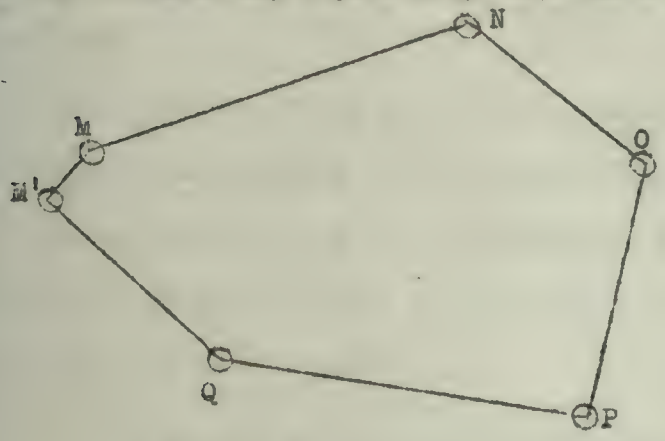
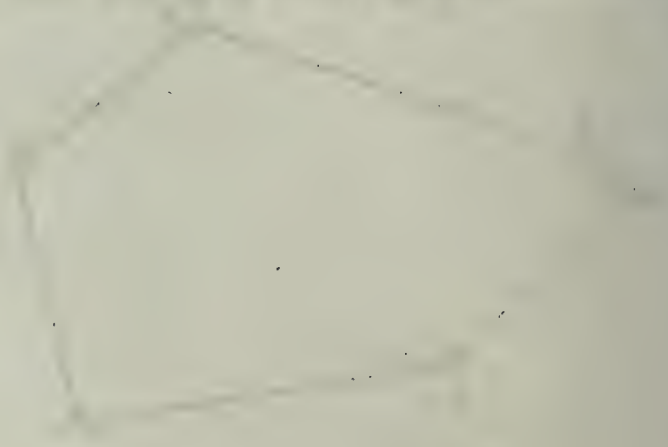


Figure 29

94) DIFFERENT WAYS OF MEASURING ANGLES

If we consider any closed figure bounded by straight lines the angles of which it is desired to measure with a transit, there are four ways in which this may be done. The method chosen is a matter of convenience, as the same results may be accomplished by any one of the four.



In traversing a polygonal figure the exterior angles may be measured one by one, which amounts to a measurement of the deflection angles in order around the field. The interior angles at each corner may be found by subtracting the exterior (deflection) angles from 180°.

In another method the interior angles may be measured directly, thus avoiding a troublesome computation; but this method is not so commonly used as the deflection method, which lends itself to progressive steps, and allows the records to be easily kept and the errors checked.

Another way to accomplish the reading of angles is to set up within the figure and take the angles in series by sighting to each station (vertex) of the polygon in order. This is best done by permitting the angular measure as shown on the graduated circle to accumulate. The closing reading will be the initial reading plus 360° - i.e. a perigon. This affords a ready, although not an absolute, check upon the accuracy of the work and in some cases this method is to be preferred to others.

The fourth way of making angle measurement is by azimuths referred to some line assumed as zero, called a reference line. This reference line may be any convenient line, as a bounding line of a figure, magnetic north and south line, starting from either the north or south point. Likewise true north and south may be made the line of reference. By convention and universal practice azimuths are always read to the right, i.e. the telescope is turned

clockwise; but either the north point or the south point (magnetic or true meridian) may be the starting point on the zero azimuth. As the azimuths are conveniently checked by means of the magnetic needle (an adjunct of every complete transit), it is well to take azimuths from the magnetic meridian. In making astronomical observations for meridian (longitude), latitude, time, etc., in solar observations or upon some star, it is best to consider azimuths with reference to the true meridian as this simplifies computations and is the general practice.

The compass needle may in this case also be used for checking. It is especially convenient when the compass plate is furnished with a variation arc to set off the declination by that means so that the "check" angle may be read directly; otherwise it will be necessary to add (or subtract) the declination for each reading of angle.

Every line has two azimuths, the forward azimuth (or simply the azimuth) taken with transit at the back end of the line, and the back azimuth taken with transit at the forward end of the line, As these two, azimuth and back azimuth, differ by 180°, the back azimuth of any line whose azimuth is known may be had by adding (or subtracting) 180° from the azimuth, or vice versa.

Orienting the instrument is an operation that is fundamental in taking azimuths. This consists in bringing the line of sight, with plates clamped (the alidade plate and its vernier preferably

PROB. NO. 11. ANGLES OF TRIANGLE 5-6-8 BY REPETITION.

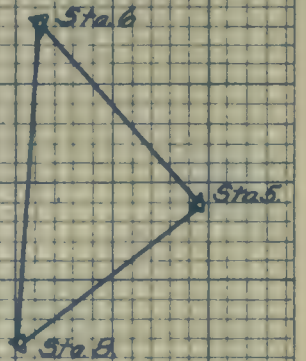
Station	Telescope	Object	Vernier A	Vernier B	Mean Vernier	Difference	Angle	Mean Angle	Corrected Angle	Remarks
Δ6	Direct	Δ5	0°00'00"	180°00'00"	00'00"					
1		Δ8	47°47'20"	227°47'20"	47'20"	47°47'20"				Single Angle
			238°56'20"	58°56'20"	56'20"	238°56'20"	47°47'16"			3 Repetitions
	Inverted	Δ5	90°00'00"	270°00'00"	00'00"					
		Δ8	137°47'00"	317°47'00"	47'00"	47°47'00"				Single Angle
			328°56'40"	148°56'40"	56'40"	238°56'40"	47°47'20"	47°47'18"	47°47'15"	5 Repetitions
Δ8	Direct	Δ6	0°00'00"	180°00'00"	00'00"					
		Δ5	43°22'20"	223°22'20"	22'20"	43°22'20"				Single Angle
			216°52'00"	36°52'00"	52'00"	216°52'00"	43°22'24"			3 Repetitions
	Inverted	Δ6	90°00'00"	270°00'00"	00'00"					
		Δ5	133°22'20"	313°22'20"	22'20"	43°22'20"				Single Angle
			306°51'40"	126°51'40"	51'40"	216°51'40"	43°22'30"	43°22'22"	43°22'19"	5 Repetitions
Δ5	Direct	Δ8	0°00'00"	180°00'00"	00'00"					
		Δ6	88°50'20"	268°50'20"	50'20"	88°50'20"				Single Angle
			84°12'40"	264°12'40"	12'40"	444°12'40"	88°50'32"			5 Repetitions
	Inverted	Δ8	90°00'00"	270°00'00"	00'00"					
		Δ6	178°50'20"	358°50'20"	50'20"	88°50'20"				Single Angle
			174°12'20"	354°12'20"	12'20"	444°12'20"	88°50'28"	88°50'30"	88°50'26"	5 Repetitions
							Total	180°00'10"	180°00'00"	Check
								180°00'00"		
							Error	00°00'10"		

Correction per angle 03⅓"

October 21, 1907
Clear and cool

Equipment:
Berger transit. 360°
Two tripods,
Two plumb bobs.
Reading glass.

Jones, L. W.
Larkin, H. E. Recorder



at zero) into parallelism with the line of reference, i.e. to zero azimuth. This orientation is, hence, the first step in measuring angles by azimuths.

There are two ways of taking azimuths with the transit. In one the back-sight is taken with telescope plunged, and the other the telescope is always normal. Both methods have their advantages and disadvantages. The first method (plunging telescope for back-sights) does not require a setting of the vernier after orientation; with the second method (telescope normal), the vernier must be re-set for each angle. This latter operation is troublesome and is likely to introduce a small error caused in setting the vernier, which may, however, be more or less compensated. The back-azimuth may always be obtained by adding 180° to the azimuth - a check that should be automatically applied whether recorded or not. If the transit is in adjustment as regards its line of collimetion and if the horizontal axis is truly horizontal and at right angles to the line of sight, the first method is preferable. (See Assignment XIV on Adjustments of the Transit.)

95) ANGLES BY REPETITION

To measure angles when great refinement is required, or when it is desired to "check" angle readings effectively, the method known as "Measuring Angles by Repetition" is resorted to. This consists in turning off the given angle between any two lines again and again from two to six times or more, and adopting the mean (average) of such readings as the true or recorded angle. To

illustrate: Suppose the angle XOY in the adjoining figure is sought. With transit at O and graduated plate clamped, vernier at zero, orient the instrument on line OX by bisecting X. Clamp lower plate, unclamp alidade plate and sight Y, carefully bisedting with tangent-screw; read angle measured. This is (approximately) the angle XOY. Now repeat the angle measurement say, five times, thus: Without disturbing the upper motion, release the lower clamp-screw and turn telescope back to X, making the necessary fine adjustment in bisecting X by means of the lower tangent-screw; unclamp the upper plate and again bisect Y. Clamp the upper plate when in that position and make the fine adjustment with the tangent-screw as before. This is the first measurement. It is immaterial whether we re-read the angle at each measurement or not; in practice it is not so read. Now by means of the lower motion sight back on X, then by upper motion sight on Y (second measurement) and so continue until the angle has been repeated five times; i.e. called six repetitions. Evidently an accumulated angular quantity six times the angle XOY (as observed at the first reading) has been turned off on the graduated plate. This accumulated quantity divided by six gives the angular value of angle XOY, and if the instrument has been manipulated with care and the points X and Y exactly bisected in each setting, the value for the angle thus obtained is much less liable to error than would be the single first reading or perhaps any lesser number than six repetitions.

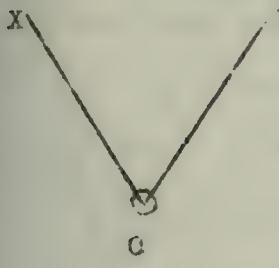


Figure 30.

Furthermore the transit and vernier are graduated to read, say to one minute of arc, but by this method it is possible to measure the angle to within a few seconds of arc, since by the accumulation of small differences, practically impossible of detection in observing the vernier, the total difference is by division distributed in the average, thus giving a more truly correct determination.

In further illustration let us assume that the angle XOY by first measurement was 23°17', by the first repeating 46°35'. Here evidently the 17' of the first determination was slightly under the mark. While at the end of the fifth repeating (for the six repetitions, as we say) the total angular quantity read upon the plate is 139° 45'; which now divided by 6 gives 23°17'30" as the most probable value of angle XOY. And this seems to have been revealed by the second reading, but it is accepted with greater assurance from the evidence of the six repetitions.

The student is urged to acquaint himself with this method of "reading angles by repetition", both theoretically and practically, as it furnishes the engineer his greatest reliance in angular measurement. It is especially useful where the degree of accuracy sought exceeds the limits of plate and vernier graduation, and is a most reliable "check" even where the greater degree of refinement is not desired.

References:

Breed & Hosmer pp. 105, 108, 109-111, Vol. I.
Tracy pp 152-156
Raymond pp. 102-105
Johnson pp. 93- 98

Exercises in Transit Use

1. Stake out a quadrangular field by pacing, no side less than 150 feet. Assume zero azimuth to be magnetic north. Set up at most westerly corner and measure the azimuth of each side by either method of taking azimuths. Record the magnetic bearings as a check against large errors in angles.

2. Lay out upon the ground a triangle, sides approximately 150 feet; not an equilateral triangle, but no angle less than 30° nor more than 120°.

Measure each angle by repetition, making six repetitions with telescope normal, beginning with A, vernier set at 0°. Then with telescope inverted obtain angle by six repetitions, starting with vernier set at 270°. Check each set of readings. Also check by taking the sum of interior angles.

UNIVERSITY OF CALIFORNIA EXTENSION DIVISION
CORRESPONDENCE COURSES IN ENGINEERING SUBJECTS

Course 1A Elements of Surveying Swafford

Assignment 11

OBSERVING FOR MERIDIAN AND LATITUDE

FOREWORD:

This assignment will treat of the several methods employed by surveyors in locating the North and South line through any position on the earth and also of determining the surveyor's angular distance north or south of the equator. These comprise the surveyor's co-ordinates upon the earth's surface, which fix his position; in relation to these the direction and course of all lines in surveying are run.

(96) PRELIMINARY CONSIDERATIONS

A few definitions and explanations are necessary at this place, in order that certain terms and fundamental facts relative to the subject may clearly be understood

Meridian is a term applied to the north and south line passing through any given point upon the earth's surface. The term is also applied to a great circle of the celestial sphere made by the intersection of a plane passing through the observer's north and south points (the north and south points of the heavens) and including the zenith and nadir. Such a plane passing through the earth determines a great circle of that body. The center of the circle is the center of the globe and the circle is the trace of the plane where it cuts the earth's sphere.

Only in changing position in a north and south line does one maintain the same meridian, as any slightest movement to east or west causes change of meridian; hence, for any position upon the earth's surface there can be one and only one meridian.

Should a surveyor move along his meridian northward, he would approach the earth's north pole, the point on the surface pierced by the axis of rotation. So also if he should proceed southward he would eventually reach the south pole. Midway between these two poles lies the equator - ninety degrees from either pole.

The Equator is a great circle upon the surface of the earth formed by a plane which passes through the center of the earth at right angles to the axis of rotation. Later it will be necessary to distinguish the earth's equator from the celestial equator, but in the present consideration the distinction is not of importance.

Since there is no natural fixed point taken in an east and west direction upon the earth's surface, a certain meridian is chosen for reference called a prime meridian, notably that passing through Greenwich, England. This is commonly used by astronomers and navigators for convenience in reckoning distances east and west and for purposes of time or longitude. Longitude and time are expressed either in degrees, minutes and seconds of arc or in hours, minutes, and seconds of time; these units being readily convertible, one into the other.

Other prim. meridians are also used, as Paris by the French; Berlin by the Germans; St. Petersburg by the Russians; etc. In the United States we occasionally make use of the meridian of Washington, D.C. as a prime reference; but since this causes some confusion, it is better for many reasons to adhere to Greenwich as the one prime meridian. We will always do so in these assignments.

Longitude is the distance either east or west of the prime meridian expressed either in degrees or hours; commonly in degrees by the surveyor and hours by the navigator. Furthermore, the longitude extends 180° on either side of the prime meridian and is expressed as so many degrees east longitude or west longitude.

Latitude is distance north or south of the equator, measured in degrees of arc on the meridian at any given place. As the equator is at quadrant distance from either pole the range of latitude is from 0° at the equator to 90° at the pole. Corresponding to terrestrial latitude is celestial (astronomical) latitude conceived as measured upon an arc of the celestial sphere. We also use the measure of polar distance or co-latitude, which is the complement of the latitude.

Since the fixing of the observer's position by means of these co-ordinates, latitudes and longitudes, is always of prime importance, we shall proceed to explain how they are determined.

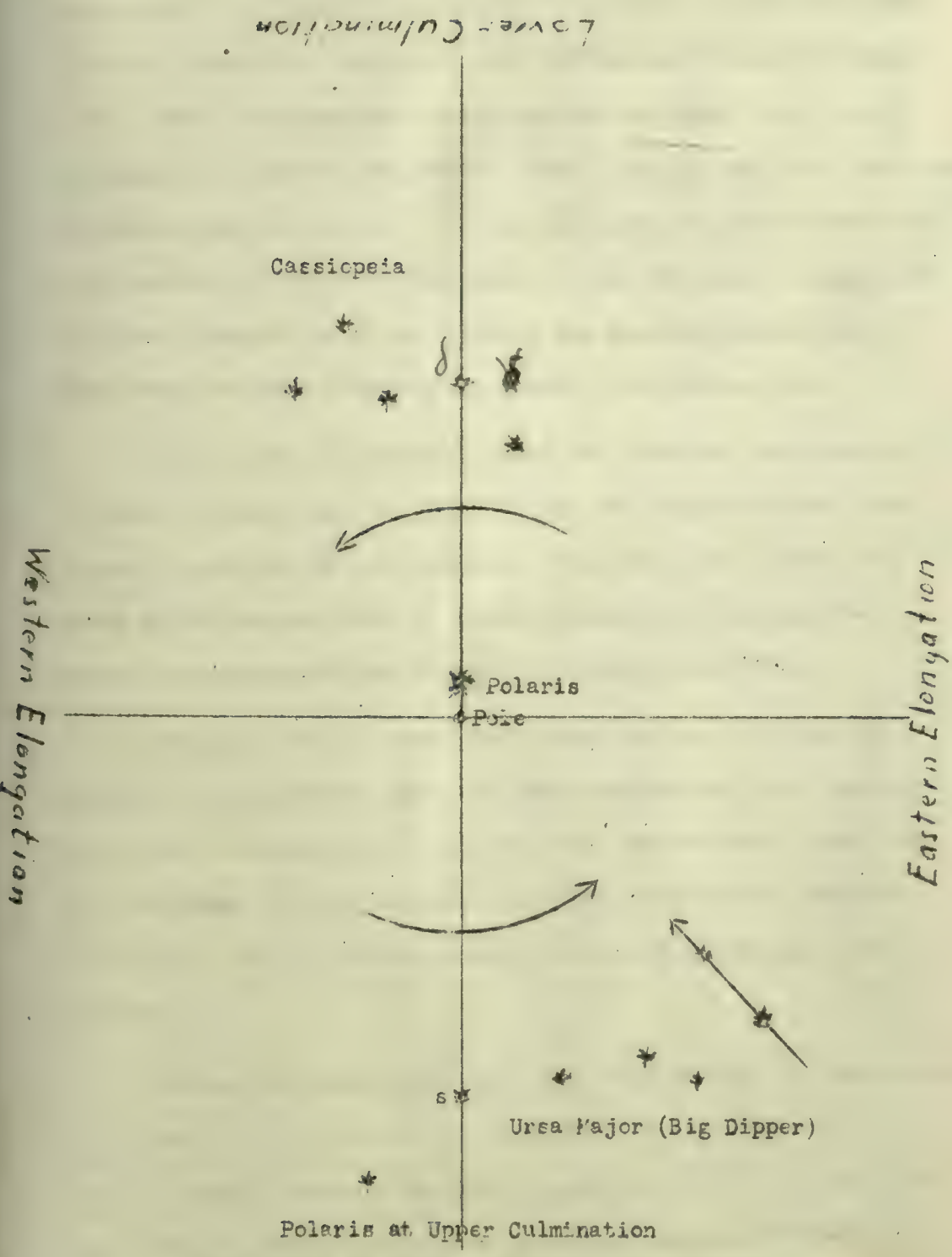
If we could sight upon the north or south pole of the earth and also lay off arc distances upon a meridian upon the earth's surface directly, we might have an ideal method of observation.

But this method is impossible. Other methods, however, accomplish our purpose even better than that of direct measurement.

The sun and the fixed stars afford us a means of determining meridian and latitude that is available at all times when the sky is clear enough to observe them. Especially favorable for this work is the sun, which may be viewed by day, and the Pole Star (Polaris), which is generally visible in thenorthern hemisphere at night. Other stars may be used, but our work will be limited to the methods of observing upon these two.

On any clear night you may see Polaris, a star of the fifth magnitude, with which all are more or less familiar. It is readily distinguished by being in line with the two extreme stars of greater magnitude in the bowl of the Dipper (Constellation of Ursa Major). On the opposite side of the Pole Star, about equidistant to the Dipper, is another constellation shaped like the letter W or Σ (Sigma of the Greek alphabet); this is the constellation Cassiopeia, and these constellations are of great convenience in locating and determining just the phase in which Polaris (the Pole Star) may be at any given time. These stars charted as here shown enable us to locate the celestial north pole, i.e. the point which the axis of the earth prolonged would pierce if extended in space indefinitely. For Polaris is quite near the pole (1°07'20") and is now approaching nearer and nearer to that position.

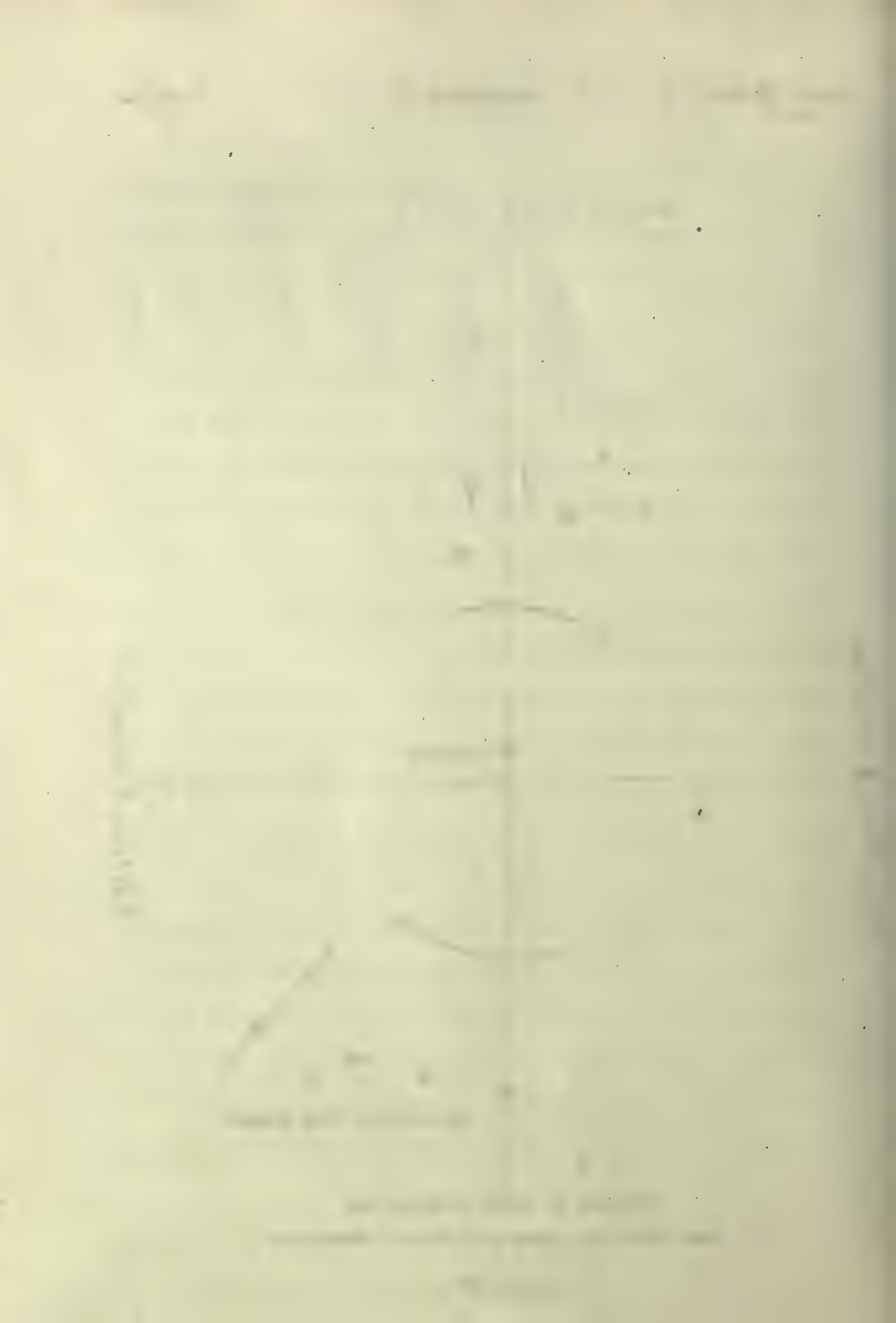
This star revolves about the pole once in twenty-four hours, and twice in that period of time it is exactly on the meridian, once



Polaris at Upper Culmination

Turn Chart to agree with Stars' Positions

Figure 31



above the pole (upper culmination) and once below the pole (lower culmination). In that time also it is once at the extreme right (eastern elongation) and once at the extreme left (western elongation). When the conditions and times are favorable it is very convenient to observe upon Polaris when in one of the four positions or phases given above, i. e., at either upper or lower culmination or at eastern or western elongation, as the necessary computations for determining meridian or latitude are somewhat simpler than those required when observation is made at any other time.

Note on the accompanying chart the relative positions of the pole, Polaris, and the stars in the two constellations, the Dipper, and delta (δ) Cassiopeiae. Hold the chart toward the north before you and turn it counter-clockwise observing the phases of culmination and elongation as thus illustrated.

Now if a line of sight be directed at the Pole Star the instant it is either at upper or lower culmination, this line of sight will evidently lie in the plane of the meridian, since such a plane passes through the earth's center, the zenith, and the north pole. Hence, the meridian or north and south line is determined.

Either the compass of the transit may be used for directing this line of sight and also for transferring this line to the earth's surface where it becomes of practical use to the surveyor. Note that the methods here to be described give one in reality the zero azimuth of Polaris with respect to the observer's north and south line upon the earth's surface.

(97) TO OBSERVE ON POLARIS WITH COMPASS AT CULMINATION

Choose an open space unobstructed by trees or buildings, extending 600 feet or more northward, on a clear night when the northern constellations are plainly visible. Let the time chosen be half an hour before the time of culmination as obtained from the Nautical Almanac or Ephemeris. (A brief list of computed times is given with this assignment. For other times you may make your own computations. See tables at close of this assignment.)

Set up the compass over a stake having a point marked upon it. Direct the line of sight through the slits in the standards and note that the star will be on the meridian (say in upper culmination) a few minutes after the line joining Sigma (s) Ursae Majoris and Delta (δ) Cassiopeiae passing through the star is vertical. The compass line of sight is now in a north and south line and should be fixed in this position. A forward point should be set in the morning light when a stake may be readily placed.

The magnetic declination may also be readily determined by reading the angle made by the needle at this time. A line joining the stakes is the true north and south line lying in the plane of the meridian.

As it is difficult to view the stars (sigma Ursae Majoris, Polaris and delta Cassiopeiae) conveniently through the standards of the compass, a modification of this method is made as follows:

Suspend a long plumb line with a heavy bob swinging in a bucket of water to prevent its oscillation. Back of this about

20 or 30 feet place a board upon two upright stakes four or five feet high and upon this board set the rear standard, removed from the compass for this purpose; bring the slit in the standard, the plumb line, and the star in line as described above and complete the operation in daylight as before directed.

At upper culmination Polaris is best seen, while at lower culmination the details of adjustments of the line of sight are more readily accomplished. Both observations are accompanied by difficulties; the observer must judge which, under given conditions, will give the best results. Much will depend upon the character of the view and upon the state of the atmosphere; if cloud or haze interferes with a clear view at lower culmination, choose the time of upper culmination instead. At certain times of year and at given latitudes one or the other of these culminations is invisible on account of daylight conditions. The hour of the night when either culmination occurs may influence one in selecting the best time.

(98) CAUTIONS IN OBSERVING ON POLARIS AT CULMINATION

As the star's apparent motion is at right angles to the plane of the meridian, it changes rather rapidly and it is, therefore, necessary to be alert, prepared beforehand, and to clamp the compass plate at the instant the star is on the meridian. Since the movement of the star is westward at upper culmination and eastward at lower culmination, it is advisable to follow the star in these directions, keeping it well in line at every point

of its path. Have everything in readiness, the time of culmination exactly computed, and your watch set to standard or mean solar time in accordance with the sort of time chosen and for which the computations have been made. To bungle your work means that you must wait 12 or 24 hours for a second observation.

(99) TO OBSERVE ON POLARIS AT ELONGATION

The transit will be used in this explanation of the method of observing for azimuth of Polaris, although the compass also could be employed. The transit is used here so that you may compare the two instruments under similar uses. It may be remarked, however, that the transit is the more satisfactory one in any case, on account of the better vision, the means for finer adjustment, and the ease with which the star may be followed and especially located at the critical moment.

Note that the star at eastern elongation is moving upward and will so appear in a transit with erecting eyepiece; with an inverting telescope the reverse movement is seen. Also at western elongation the opposite is true, the star moving downward at the extreme position to the left.

The time of eastern elongation occurs 6h 03.4m after lower culmination, while that of western elongation is 5h 54.6m later than upper culmination.

The declination (or polar distance) of Polaris is constantly changing, since the star is approaching the pole at a varying rate,

which is now about 18.8" per year. In consequence, the azimuth of the star at elongation varies. The formula for solution of the spherical triangle obtained from data of the observer's latitude (or as given in this formula the co-latitude) and the polar distance is:

$$\sin \text{star's azimuth at elong} = \frac{\sin \text{Polar dist.}}{\sin \text{co-latitude}}$$, or, expressed in logs,

log sin azimuth = log sin P.D. + co-log sin co-lat.

Tables giving the azimuth of the star at elongation (eastern and western) are compiled, a brief table of this nature being appended to this assignment, and are a ready means of obtaining these figures without labored computation. The engineer in practice should avail himself of such helps, but since the student must master principles, he should forego the use of ready-made stuff to the detriment of sound training.

(100) DIRECTIONS IN DETAIL FOR USE OF THE TRANSIT IN THIS OBSERVATION AT ELONGATION

A short time before the star reaches, say eastern elongation, set up the transit in a suitable place with an open view extending 600 feet or more to the northward. Have at hand a ready means of illuminating the cross hairs in the transit, and by shortening the telescope, focus the objective and adjust the eyepiece upon Polaris and see that there is neither parallax nor aberration due to improper adjustment of the telescope. Carefully level the transit plates and clamp the alidade plate, setting the A vernier to zero, using the tangent screw. Also have the clamp for the vertical

motion lightly set so that the telescope may be readily transited (plunged). The vertical cross hair is the one used in this observation and should of course be in perfect vertical adjustment. As the change in azimuth is very slow compared with such change at culmination, there will be ample time to make the two sightings, the first with telescope normal, the second with plate reversed and telescope plunged, thus eliminating any slight lack of adjustment. Be careful in both positions to see that the star appears to "thread" the vertical cross hair, which will be with an upward movement in the erecting instrument and with a downward movement in the inverting instrument at eastern elongation, the case we have chosen for these directions. In reversing for the observation with telescope plunged the graduated plate is unclamped and when the star is again bisected, or brought to the vertical cross hair, the final adjustment is made with upper plate tangent screw. If the B vernier now reads exactly zero (that is, A has moved through just 180°), the telescope has pointed to the stars at elongation in both positions. If there is a difference in the two readings, A vernier and B vernier, then one half of the difference must be added (or subtracted) from the A zero when the telescope is returned to normal position. Thus the instrumental correction has been set off on the limb and if now the azimuth, taken from the table or computed from the formula given above ($\sin \text{azimuth} = \frac{\sin \text{Polar dist.}}{\sin \text{co-latitude}}$) be turned off on the limb to the <u>right</u> - the telescope moving to the <u>left</u> - the line of sight is <u>pointing in the plane of the meridian</u>.

To establish the meridian line on the earth, line in a forward stake 600 or more feet away and also set another stake under the plumb bob point of the transit. These two points determine the meridian line.

The cross hairs may be conveniently illuminated by holding an electric torch or a bulls-eye lantern reflecting into the objective tube. To effectively do this, a piece of glazed paper or tracing cloth through which a central opening half an inch in diameter may be mounted upon the objective end of the telescope; set this at an angle of approximately 45° and the light held at the side will be reflected downward through the tube.

(101) CAUTIONS IN OBSERVING

In general the cautions given for observing at culmination apply in this case. In addition to these see that the cross hairs are distinct and that the star is sharply defined in the field of view; also that the reflector described above does not interfere with a clear view. When once sighted the star should be kept in view and followed as closely as possible by means of the tangent screws both in azimuth and altitude. As before stated the star at about the time of elongation does not change in azimuth perceptibly for some minutes, but when the least change is noticed, cease to follow the star as it is then no longer at (or near) elongation.

(102) TO FIND THE LATITUDE OF THE OBSERVER

It will be noted that at the equator the pole of the heavens is on the horizon; also as the observer moves northward the altitude

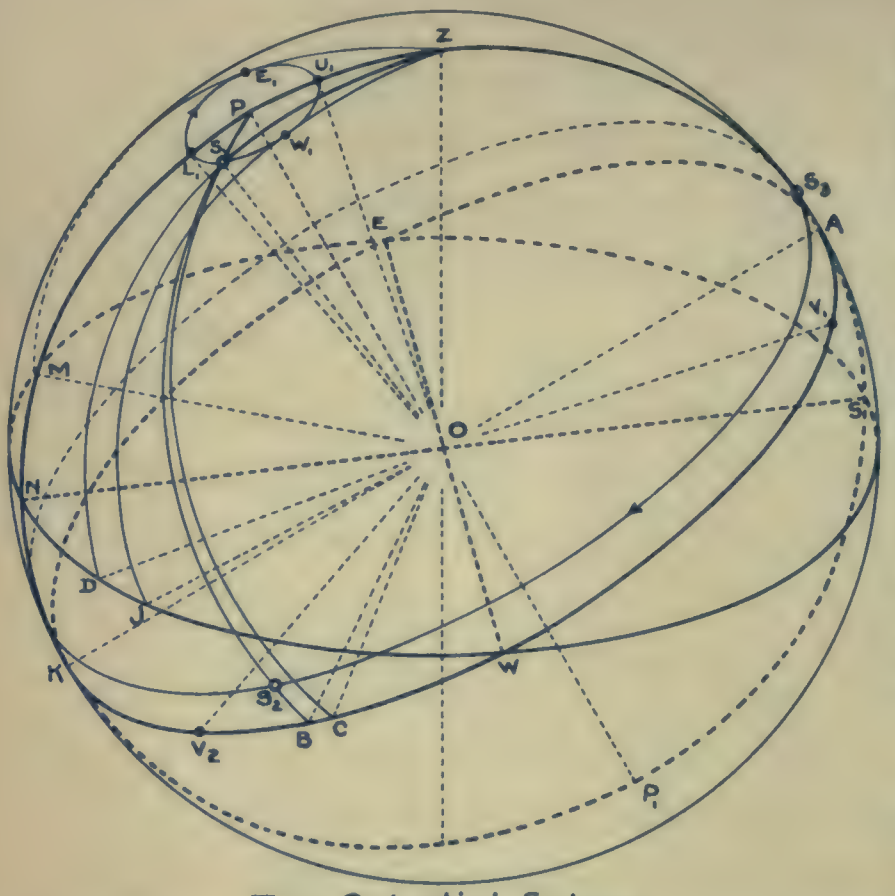
of the pole increases in exact proportion with the latitude; at the north pole of the earth the celestial pole is in the zenith; hence the latitude at any point on the earth's surface is equal to the altitude of the celestial pole.

Therefore, with the vertical circle and level on telescope, in perfect adjustment (See Assignment XIV, Adjustments of the Transit) bisect the Pole star at culmination , either upper or lower. Read the angle of elevation on Vertical Circle. If at upper culmination, subtract the polar distance of the star and correct for refraction in altitude as given in the table appended to this lecture. If at lower culmination add the polar distance and correct for refraction in altitude as before. The net result in either case is the latitude of the place.

(103) CAUTIONS IN OBSERVING FOR LATITUDE

Choose the time for beginning of the observation half an hour before culmination and follow the star for a few minutes at least before that time. Note that whereas the star changes rapidly in azimuth at culmination, on the contrary it moves very slowly in altitude at these times and for several minutes before and after culmination has (approximately) the same altitude. Therefore, without too great presumption, the observer may use considerable deliberation in sighting the star for latitude. But atmospheric refraction causes the star to appear higher above the horizon than it is in reality; hence, do not neglect to reduce the observed altitude by the amount of the refraction. Note that the refraction

To Determine the Azimuth of Polaris at Any Given Standard Time
To Compute Angles AOC and NOD.



The Celestial Sphere.

Definitions.

NESW	Plane of the observer's meri
NS	Observer's meridian
Z	Observer's zenith.
POP,	Polar axis.
KV₂WAE	Plane of the equato
L=PON=ZOA	Observer's latitud
EW	East and west line
U,W,S,L,E,	Path of Polaris.
U,	Polaris at upper culm
W,	Polaris at western elo
L,	Polaris at lower culm
E,	Polaris at eastern elo
δ=L,OK=U,OA	Declination of Pola
POL,=POU,=POS,	Polar distance of Po
SOJ	Azimuth of Polaris at W.
SOM	Azimuth of Polaris at E.
S,	Polaris as observed at h
SOD	Azimuth of Polaris as obs
S_3	Mean Sun at observer mean noon.
V,	Vernal equinox at obs local mean noon.
V_2	Vernal equinox when Po
S_2	Mean sun at local hou when Polaris is at
AOB ÷ 15	Local mean time when
AOV_2	Observer's sidereal h when Polaris is at
$V_2OA \div 15$	Observer's sidereal t ascension of observe when Polaris is at
$V,OA \div 15$	Right ascension of m at observer's local m
$V_2OC \div 15$	Right ascension of Po
t = AOC =	V_2OA minus V_2OC. Hou of Polaris at S_1.
b = NOD	Bearing of Polaris a S_1.

Note. Right ascension is measured from the vernal eq in the direction WAEK up to 24 hours. Hour ar measured from the South, A, in the directio AW up to 24 hours.

Known Data.

1. From the observer's watch: Standard time of observation of Polaris at S_1 i.e. hour angle for standard meridian corresponding to AOB for observer's meridian.
2. From the nautical almanac: Right ascension of the mean sun at the previous Greenwich mean noon. This corresponds at the Greenwich meridian to the sidereal hour angle V,OA for the observer's meridian.
   Right ascension of Polaris at S_1.
   Declination of Polaris.
3. From a U.S.G.S. map or by observation: Observer's latitude.

Solution.

Greenwich sidereal time at Greenwich mean noon equals the right ascension of the mean sun at Greenwich mean noon.

Observer's sidereal time at observer's local mean noon equals right ascension of mean sun at previous Greenwich mean noon plus increase in right ascension during the mean time period between the previous Greenwich mean noon and the observer's local mean noon. Observer's sidereal time when Polaris is at S_1 equals the above sidereal time plus the further increase in right ascension during the mean time period between the observer's local mean noon and the observer's local mean time of observation plus local hour angle AOB.

Sidereal time gains on mean time 9.8565 seconds in one mean time hour. Observers local mean time differs from observer's standard time by difference in longitude between the standard and the observer's meridian divided by 15; it is earlier than standard time when the observer is west of the standard meridian.

The hour angle of Polaris equals observer's sidereal time minus the right ascension of Polaris.

The azimuth of Polaris at S_1 is found by solving the spherical triangle PZS, for the angle b at Z.

$$\tan b = \frac{\sin t}{\cos L \tan \delta - \sin L \cos t}$$

When t is smaller than 180° Polaris is west of the meridian; when greater than 180°, Polaris is east of the meridian.

Example.

Find the azimuth of Polaris on April [illegible] at 9:30 P.M., Pacific Standard Time [illegible] Berkeley, California. Latitude, N. 37° [illegible] Longitude, 122° 15' 42" W. = $8^h\ 9^m\ 2[illegible]^s$

Right ascension of mean sun at Greenwich mean noon on Apr. 9-1916 ..... $1^h\ 9\ 05$
Local time of observation is standard time of 8h meridian minus $9^m\ 3^s$ .. $9^h\ 2\ 57$
Increase in right ascension in the period between Greenwich mean noon and the local mean time of observation equals * 17.50 x 9.8565 ..... h [illegible]
Sidereal time when Polaris is at S_1 ..... 10 [illegible]
Right ascension of Polaris when observed at S_1 ..... h [illegible]
Hour Angle of Polaris at S_1 ..... 9^h [illegible]
t = 136° 11' [illegible]
Declination of Polaris at S_1 ..... + 88° 51' [illegible]

log cos L = 9.897281	log sin L = 9.71 [illegible]
log tan δ = 1.701266	log cos t = 9.833 [illegible]
1.598547	9.6 [illegible]
39.6777	- 0.430
- 0.4430	
40.1207	
log sin t = 9.840295	
log 40.128 = 1.603368	
log tan b = 8.236927	

b = 0° 59' 19" west of north

Required azimuth of Polaris is 179° 00' 41"

* 17.50 = $8^h\ 9^m\ 3^s$ plus $9^h\ 20^m\ 57^s$

is greatest near the horizon and decreases toward the zenith; the correction for refraction in altitude will always be less at upper culmination (when the star's observed altitude is greatest) and greater at lower culmination (when the observed altitude is least). The transit used in this work should be equipped with a full vertical circle and the angle read both in normal and inverted position of telescope to eliminate graduation error. The mean of these twp readings is to be taken as the observed altitude. This with the corrections for polar distance and refraction give the latitude as the net result.

References

Breed & Hosmer, pp. 212 - 220, Vol. I.

Tracy, pp. 354 - 360

Raymond, pp. 89 - 93

Johnson, pp. 30 - 36

As exercises in connection with this assignment it is recommended that you make such observations as the means at your command permit. You may study the heavens to locate Ursa Major and Cassiopeia and especially by means of these to locate the North Star.

An instructive exercise would be the determination of the meridian by the plumb-line method, which is possible without expensive apparatus. In addition to this the magnetic declination may also be determined.

Questions.

1. The declination of Polaris on July 1, 1921 is $88^{\circ}52'46''$, what is the azimuth of the star at elongation for latitude $37^{\circ}31'$ North?

2. Find the times of upper and lower culmination of Polaris on November 10, 1921.

TABLE I

AZIMUTH OF POLARIS AT ELONGATION, 1921

Latitude	Azimuth	Latitude	Azimuth
30°	1°17.'2	40°	1°27.'3
31°	1 18. 1	41°	1 28. 5
32°	1 19. 0	42°	1 29. 9
33°	1 19. 8	43°	1 31. 4
34°	1 20. 7	44°	1 32. 9
35°	1 21. 7	45°	1 34. 6
36°	1 22. 8	46°	1 36. 3
37°	1 23. 8	47°	1 38. 1
38°	1 24. 9	48°	1 40. 0
39°	1 26. 1	49°	1 42. 0
40°	1 27. 3	50°	1 44. 1

TABLE II

TIME OF CULMINATIONS OF POLARIS

Meridian of Greenwich - Mean Time - Civil Date

1921	Upper Culmination				Lower Culmination			
	h	m	s		h	m	s	
July 1	6	57	33	A.M.	6	55	43	P.M.
July 15	6	02	46	"	6	00	56	"
Aug. 1	4	56	15	"	4	54	25	"
Aug. 15	4	01	27	"	3	59	37	"
Sept. 1	2	54	51	"	2	53	01	"
Sept. 15	1	59	59	"	1	58	09	"
Oct. 1	0	57	13	"	0	55	23	"
Oct. 15	0	02	14	"	0	00	24	"
Nov. 1	10	51	28	P.M.	10	53	18	A.M.
Nov. 15	9	56	21	"	9	58	11	"
Dec. 1	8	53	19	"	8	55	09	"
Dec. 15	7	58	05	"	7	59	55	"

Difference for one day: 3.7 minutes

...	...	...	...
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UNIVERSITY OF CALIFORNIA EXTENSION DIVISION
CORRESPONDENCE COURSES IN ENGINEERING SUBJECTS

Course 1A Elements of Surveying

Assignment 12

SOLAR OBSERVATIONS - SOLAR INSTRUMENT

FOREWORD:

This assignment will first deal with the necessary astronomical phenomena preliminary to methods of solar observations for meridian, latitude, and time; following these will be taken up the solar attachment and its use in solution of the spherical triangle for determining the observer's meridian; and finally, the methods of direct observations on the sun for azimuth, latitude, and time.

(104) THE SUN AND ITS MOTIONS IN THE HEAVENS

In its motions in the celestial sphere the sun passes through certain unique positions and appears to an observer upon the earth to traverse a somewhat peculiar path, changing in its equatorial distances, its time of rising, of culmination (or transit over any meridian), and of setting.

The cause of these seemingly irregular movements is due to the fact that the earth's axis is inclined at an angle (23°27' nearly) to the ecliptic or the plane of the path in which the earth revolves around the sun. While the earth in its diurnal rotation and annual revolution is passing through these two cycles of change, the observer upon the earth's surface sees the sun as though it were moving upon the celestial sphere. Hence the sun appears to

rise at a different point on the eastern horizon, set at a different point on the western horizon, and pass from rising to setting over a slightly different path each day.

The explanation is probably better made by saying that the plane of the ecliptic and the plane of the earth's equator are inclined to each other by the angle of 23°27' (nearly). Since these planes are so inclined they intersect each other and the two points upon the equator (celestial equator) cut by the ecliptic are called the equinoxes, because when the earth is at these points, the days and nights are equal, the sun rising at 6 a.m. and setting at 6 p.m. The vernal equinox is the name given to the one which takes place about the 21st of March; while the autumnal equinox occurs about the 22nd of September. Twice a year the sun seems to reach a most northerly and most southerly point in its course, and these are designated the summer solstice (June 21st) and the winter solstice (December 22nd) respectively.

So in March the sun appears on the equator, and in June about 23°27' northward, the change being slow, from day to day. The angular distance measured upon the great circle (a meridian circle) passing through the sun and reckoned from the equator, is called the sun's declination. The declination may be either north (+) or south (-), its magnitude varying from 0 to 23°27'. From June to September its north declination is decreasing, reaching 0° at September 22nd; then changing to south declination it increases until Dec. 22nd; and finally decreases in south declination until it again reaches the equator at the time of the vernal equinox.

The complement of the declination is the polar distance; 90° - d = P.D. is the simple equation expressing this relation, and, as the declination is positive (+) when north and negative (-) when south, the P.D. varies between 90° - 23°27' and 90° + 23°27', i. e. between 66°33' and 113°27'.

The sun's position is also determined by its altitude above the observer's horizon, or rather the complement of the altitude (90° - altitude) which is the zenith distance.

The azimuth of the sun is the arc on the horizon measured between the meridian and the vertical circle through the sun. Reckoned from the south point of the horizon, generally, the azimuth is 180° greater than when taken from the north point.

The right ascension is the sun's distance in arc from the vernal equinox measured eastward upon the celestial equator. Another co-ordinate called hour angle is the arc of the equator measured westward from the meridian to the hour circle through the sun. These two arcs are conveniently expressed in time units, hours, minutes and seconds, but may readily be reduced to degrees, minutes, and seconds (°, ', ") as occasion requires. Right ascension is the hour angle reckoned from the vernal equinox.

From the foregoing considerations we have three systems of co-ordinates, as follows:

(1) The Horizon system employing altitude and azimuth; (2) the Equatorial system, which makes use of the declination and hour

PLANE SURVEYING

COURSE X–1A

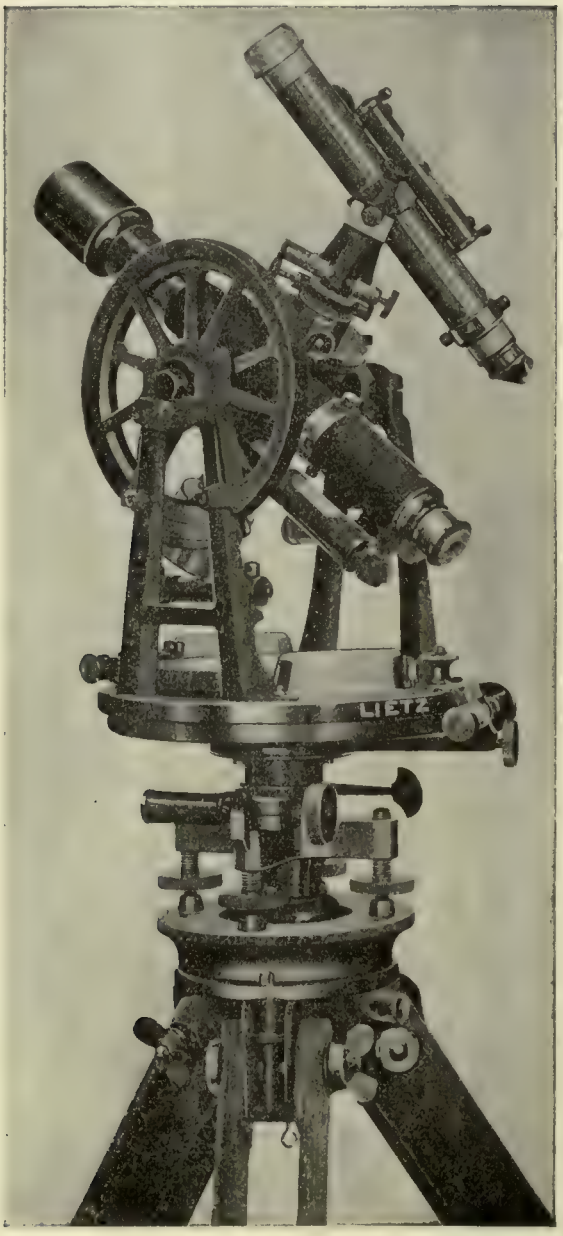


PLATE VIII

ENGINEERS' TRANSIT FITTED WITH
SAEGMULLER SOLAR ATTACHMENT

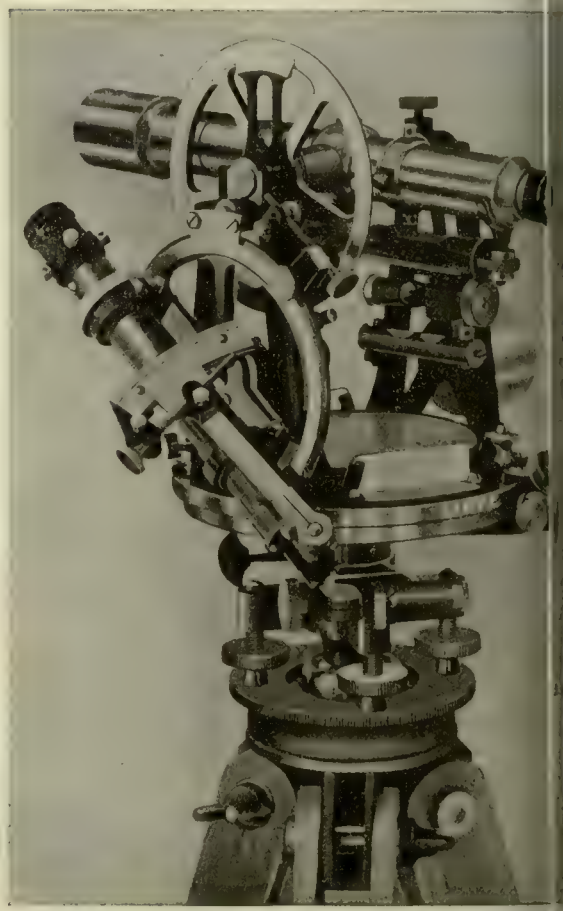


PLATE IX

MOUNTAIN AND MINING TRANSIT
FITTED WITH
SMITH SOLAR ATTACHMENT

THE CELESTIAL SPHERE

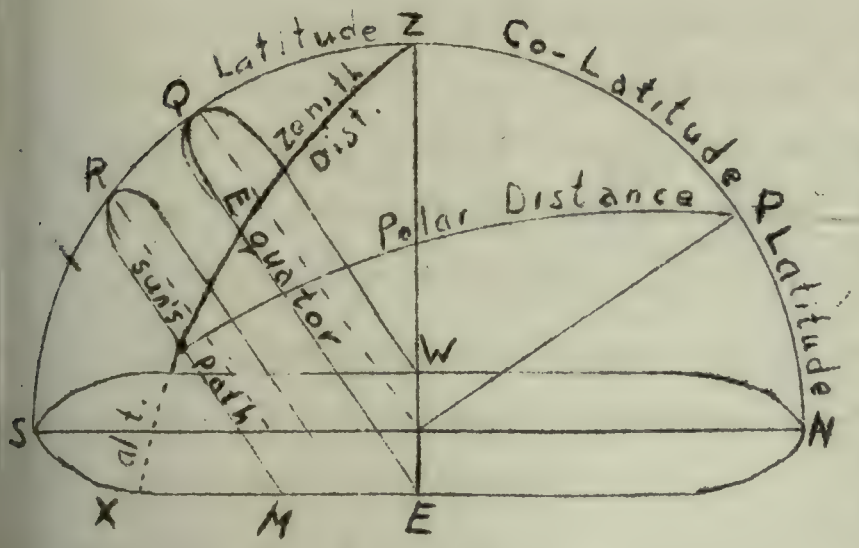


Fig. 32

O, the earth at center of sphere

ESWN Plane of horizon

SQZPN Meridian of observer

EQW Plane of equator

MRT Sun's path

Angular distance of sun from equator at any time is sun's declination, which is continually changing.

Alt. Sun's altitude

Sun - Z Zenith distance

PN = QZ Latitude

ZP = 90° - QZ Co-Latitude

Zenith distance is the complement of the altitude, i.e., 90° - alt.

Co-Latitude complement of latitude - i.e., 90° - Lat.

Polar distance complement of declination - i.e. 90° - Declination. And, since the declination may be either +, when north, or -, when south, this arc (Polar Dist.) will vary between 66°33' and 113°27'.

Arc NX, the measure of angle, Sun-ZP, is the sun's azimuth.

Arc of equator QY, measure of angle Sun-PZ, is the sun's hour-angle.

Refraction Corrections for Altitude

Alt.	Ref.	Alt.	Ref.	Alt.	Ref.
10°	5'16"	18°	2'56"	30°	1'40"
11	4 48	19	2 46	35	1 22
12	4 24	20	2 37	40	1 09
13	4 04	21	2 29	45	0 58
14	3 47	22	2 22	50	0 48
15	3 32	23	2 15	60	0 33
16	3 18	24	2 09	70	0 21
17	3 07	25	2 03	80	0 10

angle (or Right Ascension); and (3) the Terrestrial system, using the latitude and the longitude, which define the observer's position upon the earth. This last is the one which must be determined by the surveyor, and hence all observations of an astronomical nature follow this system.

By reference to the diagram (Fig. 32) showing the celestial sphere, with the accompanying explanations, the foregoing definitions will be made more intelligible.

(105) DETERMINATION OF MERIDIAN

The solar attachment must be carefully adjusted (see Assignment XIV for detailed directions for these adjustments).

Having determined the declination settings for the times of observation, bring the solar telescope and the transit telescope into the same vertical plane by sighting both upon the same point.

(a) Set off the declination for the time of observation on the vertical circle with transit telescope pointing southward; if the declination is north, depress the telescope; if south, elevate it.

(b) Clamp the transit telescope in this position, and level the solar telescope; the solar is now in the horizon and the angle between the solar and the transit telescopes is that of the sun's declination. The angle formed by the polar axis and the solar telescope will be the sun's polar distance (90° ± declination).

(c) Unclamp the horizontal axis of the transit telescope

and set off the co-latitude of the observer's position on the vertical circle. The latitude may be taken from a map or may better be determined by a previous observation for latitude. The polar axis now points to the Pole.

(d) With the lower motion of the transit and that of the solar unclamped, (if in the northern hemisphere, pointing in a southerly direction) the two telescopes may now be turned, each about its vertical axis by means of the tangent screws, until the sun can be followed in its path, its image falling symmetrically upon the field of the solar, thus:

As the sun changes less than one minute of arc per hour, the time is ample for making the setting of the sun's image and may be followed in this phase for several minutes.

The position of the combined instrument is now as follows: The Polar axis points to the Pole; the solar telescope to the sun; the transit telescope is in the plane of the meridian, and the triangle, Sun-ZP, has been mechanically solved.

Set a stake beneath the transit and a small tack exactly at a point A, defined by the plumb bob. Depress the transit telescope and at a point B, 600 feet or more distant, set another stake with a tack center. Plunge the telescope and at several hundred feet to rear set still another stake, C, with tack center; if the line of collimation of the transit telescope is at right angles to the horizontal axis, the line BAC will be a straight line coinciding with a meridian on the earth. From this line the true azimuth of any line may now be determined.

Also observe the position of the magnetic needle for magnetic declination.

Should it be desired to determine the true azimuth of any line passing through the point occupied by the transit, read the limb and then open the upper motion, sight upon a point in the line, and again read the limb. Or, more conveniently, if the A vernier is set to zero on the limb, then the azimuth angle may be read directly. Should it be desired to determine the azimuth angle accurately, employ the method of repetition, using the points B (to the south for south azimuths) and C (to the north for north azimuths) as points of reference.

(106) LATITUDE OBSERVATION

The meridian having been laid out, the transit may be easily brought again into the same meridional position and the altitude of the sun taken at apparent noon (i.e. when the sun is on the meridian). Then having computed the declination for the day and hour (apparent noon of the date of observation) you may compute the latitude. The following formula is applicable:

Lat. = 90° - h $\pm$ d, in which h is the altitude of the sun's center, and d is the sun's declination. The value of h must be corrected for the semi-diameter of the sun (approximately 16') when observing for altitude upon either upper or lower limb, subtracting this correction from h if the upper limb is used, adding the correction to h if the lower limb is used. Also d must be corrected for refraction, added when sun is north and subtracted when

sun is south of equator. (This applies for observations made in the northern hemisphere.) Furthermore the altitude of the sun's center should be corrected for refraction in altitude; or this last correction may be applied to the computed latitude. As refraction always causes the apparent altitude to be greater than the true altitude, the sun's apparent polar distance will always be less than the true polar distance.

(107) DIRECT OBSERVATION ON SUN FOR MERIDIAN

Compute the declination at time of observation and from this obtain the polar distance, which is equal to 90° - d (d is + at north declination, - at south declination).

Observe the sun for altitude, making correction for semi-diameter and for refraction in altitude, and from this obtain the zenith distance, which is equal to 90° - alt.

The latitude (from a good map or previous observation for latitude) subtracted from 90° gives the co-latitude l, (co-lat. = 90° - lat.)

Let S = 1/2 (P.D. + co-lat. + Z.D.), then

$$\cos 1/2 \text{ sun's azimuth} = \sqrt{\frac{\sin s \cdot \sin (S-P.D.)}{\sin \text{co-lat.} \cdot \sin Z.D.}}$$

which lends itself readily to logarithmic computation; thus log cos 1/2 A = $\frac{1}{2}$(log sin S + log sin (S-P.D.) + colog sin co-lat. + colog sin Z.D.)

(108) TO OBSERVE TIME WITH SOLAR

Make the observation for meridian as above directed. Note the time on watch at the instant that the sun is centered in field

AZIMUTH NO. 18 AZIMUTH OF A LINE BY DIRECT OBSERVATION ON SUN

(23)

Telescope	Limb of sun	Hor. Angle	Vert. Angle	Time
Normal	☉	34° 06.0	50° 06.0	2h 03m
Inverted	☌	35° 39.0	49° 13.0	2h 05m
Inverted	☌	36° 18.0	48° 54.0	2h 07m
Normal	☉	36° 16.0	49° 03.0	2h 10m

Equipment.- April 8, 1919.
Wi & E. Transit #30305 Clear & warm.
Plumb-bob
Reading Glass
Watch Swafford, P.A. & Speasa, J. Time

Observation at Berkeley, Calif.
Longitude 122° 15' 4" W. Latitude 37° 52' 24" N.
Time is P.M. Sun west of meridian
Angles from mark to sun measured clockwise.
Angles read to nearest 0.5 minute. Time to nearest minute. Initial vernier reading 0° 00.0'.

= COMPUTATION =

Observation No.	1.	2
Av. Horiz. Angle	34° 52.5	36° 17.0
Av. Vert. Angle	49° 39.5	48° 58.5
Av. Time	2h 04.0m	2h 08.5m
Obs. Altitude	49° 39.5	48° 58.5
Refraction	- 0.8	- 0.8
True Altitude	49° 38.7	48° 57.7
G.M.T. Obs.	10h 04.0m	10h 08.5m
Decl. G.M.N.	+6° 54.7	+6° 54.7
Change in decl.	+ 9.4	+ 9.5
Declination	+7° 04.1	+7° 04.2
log sin d	9.09009 -10	9.09019 -10
log sec h	0.10272	0.10272
log sec h	0.18881	0.18262
log n	9.38162 -10	9.37553 -10
n	0.24078 (+)	0.23744 (+)
log tan L	9.89083 -10	9.89083 -10
log tan h	10.07073 -10	10.06025 -10
log m	19.96156 -20	19.95108 -20
m	0.91530 (-)	0.89348 (-)

Formula

$$\cos Z = \frac{\sin d}{\cos L \cos h} - \tan L \tan h$$

In computations $\frac{\sin d}{\cos L \cos h} = n$ $\tan L \tan h = m$

d = declination h = altitude
L = Latitude Z = Azimuth of sun

= COMPUTATION - cont. =

m	0.91530 (-)	0.89348 (-)
n	0.24078 (+)	0.23744 (+)
Cos Z	0.67452	0.65604 (-)
Z	47° 35.0	49° 00.1
Mark to sun	33° 02	36° 17.0
Mark to Merid	12° 42.5	12° 43.1

Azimuth of line = 12° 42.8 from S.

of solar and clamp the solar telescope.

Set the forward stake with a tack center by pointing the transit telescope without changing the solar. Return the transit telescope to a horizontal by means of its level, and clamp the motion. The polar axis now points to the zenith. By inclining the solar telescope set a stake with a tack center bisected by the vertical cross-hair at the middle of the solar field. This stake should be 200 feet or more distant.

Then by a use of the transit limb, measure the angle between the meridian previously set and the last mark. This is the hour angle or Right Ascension in °, ', ", which must be reduced to time units, hrs., mins., secs.

Compare the time thus obtained with the watch and set the latter fast or slow as required; or, if it is not desirable to change the watch, this difference may be noted as a correction for all time observations. If the watch time is the so-called "standard" time, this must be changed to local mean time before comparison is made.

QUESTIONS:

1. (a) Change 14º 37' 42" to time measure.
   (b) Change $3^h\ 40^m\ 15^s$ to arc measure.

2. An observer in longitude 124° 07' 58" notes the time of his watch is 8:43-1/4 A.M., Standard Time. Required: The Mean local time of the observation.

REFERENCES:

Tracy	Appendix I, page 620
Raymond	Pages 116 - 126
Johnson	Pages 99 - 102
Breed & Hosmer	Vol. I, pages 63 - 70

UNIVERSITY OF CALIFORNIA EXTENSION DIVISION
CORRESPONDENCE COURSES IN ENGINEERING SUBJECTS

Course 1A Elements of Surveying Swafford

Assignment 13

ADJUSTMENT OF INSTRUMENTS

FOREWORD:

This assignment will deal with the usual adjustments of surveying instruments, special attention being given to those made in the field. The methods of detecting instrumental errors of the more serious kinds - such as would require the skill and appliances of the instrument maker - and of correcting them, will also be pointed out. As references will be made to the various sections of this assignment in the matter of instrumental adjustments, a simple system of numbering the several paragraphs has been carried out. The scheme will be quite evident after a few moments of study.

(109) (A) SPIRIT LEVEL

(1) Adjustment depends upon a principle called reversion. If the bubble-tube is placed longitudinally parallel to a truly horizontal line and the position of the bubble noted, the bubble should have the same position if the bubble-tube is reversed through 180°. In Figure 33 the bubble-tube has the position MN

Figure 33

Figure 34

on the horizontal line AB; the bubble is at O, the middle of the tube. In Fig. 34 the line AB remains unchanged, but the bubble-tube is reversed through 180° to the position NM; the bubble should now

be in the same relative position in the tube, i.e. at O. If the bubble-tube were revolved upon a horizontal plane through 360°, the relative position of the bubble would not change.

(2) Experiment may be made by the student with a carpenter's common level, as follows:

Joint up the edge of a piece of board about 6 inches wide by 3 feet long, so as to have a true, fiducial edge. Clamp lightly in a bench-vise and, by placing the level upon it, bring it to a horizontal position in the vise. Reverse the level. Does the bubble stand at the middle of the tube as before? If not, what must be done with the board in the vise to cause the bubble to return to the mid-position? Do this, and again reverse the level to the first position. What do you notice regarding the bubble now? Alter the slope of the board in the vise so as to cause the bubble to approach half-way to mid-position, and reverse the level. You will now note that the bubble will occupy the same relative position in the tube no matter which direction the level has with respect to the straight edge. Hence:

(3) To adjust a carpenter's level.

Use a straight edge as in the above experiment; level up carefully; reverse the level through 180°; bring the bubble half-way to center by tilting the straight-edge in the vise; correct the setting of the bubble-tube in the level-block. In the adjustable level, a screw acting at one end of the brass protecting plate enables one to raise or lower that end of the bubble-tube.

Some levels require the protecting plate to be removed, revealing the adjusting screw beneath. Still other levels, those of the cheaper kind, have the tube imbedded in cement (plaster of Paris) and are not adjustable.

The principle of reversion is applied in making level adjustments on all surveying instruments and will frequently be referred to in what follows respecting adjustments. (See A-1, 1st adjustment.)

(110) (B) ADJUSTMENTS OF THE SURVEYOR'S COMPASS

(a) The needle in the compass should be (1) Balanced on the pivot, (2) straight, that is in the same vertical plane through the pivot, (3) well-magnetized so that it responds promptly to the earth's action.

(1) To balance the needle, level the plate carefully and observe whether the ends of the needle are equally disposed with respect to the graduated circle; if not, move the coil of fine wire on the south end so that the needle balances. To do this it is necessary to remove the glass cover and take the needle from the box.

(2) To straighten the needle. By observing both ends of the needle it is possible to determine, first, if the needle is straight; second, if the pivot on which it turns is in the line of sight of the two standards. If the pivot is at the center of the graduated circle, and the needle straight the readings of the opposite ends will show a constant difference of 180°. If the

needle is concave to the right, the right hand segment of the circle will be less than 180° and consequently the left hand segment greater than 180° for every position of the needle. In such case remove the needle and support it near its ends, laying it upon the concave side, and press it gently to make it straight. Test it frequently by returning it to the pivot and observing if it is straight. If the needle is straight and the pivot is not at the center of the graduated circle, the needle reading of its two ends will differ by exactly 180° in only two positions, which will be themselves 180° apart. All other readings than those just mentioned will differ from 180°, one segment being less than 180°, the other greater than 180°. To adjust the pivot in such case, determine the line of no difference, remove the needle and carefully press the pivot in the direction of this line forward or backward so that the needle will give readings that differ by 180°, when at right angles to the line of no deviation. The pivot having been adjusted and the needle straightened, adjustment (3) is made.

(3) To Magnetize the Needle

Remove the needle from the box and support it throughout its length; stroke it with a bar or horse-shoe magnet, applying the north end of the magnet to the south end of the needle, or the south end of the magnet to the north end of the needle. The needle can also be quickly magnetized by bringing it into contact with the poles of a dynamo or motor while current is passing through them. Of course these must be of the direct current (D.C.) type, and the

needle placed in position with south end at + pole and north end at - pole of the motor or generator. If a mistake has been made at first trial, the error may be corrected by reversing the needle in a second trial and allowing it to remain in contact long enough to remedy the fault. The following conditions should now obtain:

1) The standards should be vertical when the compass plate is horizontal.

2) The line of sight should lie in a plane passing through the slits in the two standards and including the central pivot on which the needle swings and the exact north and south points of the graduated plate. These matters require the attention of the maker.

(111) (C) DUMPY LEVEL

(1) To adjust the horizontal cross-hair to true horizontality, sight on some well defined point at a distance of about 50 feet, setting one end of the horizontal cross-hair upon it. Turn the telescope about the vertical axis and if the point remains on the cross-hair, the latter is horizontal; if not, it requires adjusting. To do this, loosen the four capstan-head screws that hold the cross-hair reticule, turn the latter to right or left as the case may be until the distant point is made to "thread" the horizontal cross-hair throughout its length. This adjustment is then complete. (See Print A-1, Preliminary.)

(2) To collimate the telescope, unscrew the plate which holds the pinion operating the object tube. This will permit the tube containing the optical parts to be rotated. Now sight upon a

distant point, rotate the tube, and observe the position of point of intersection of cross-hair with respect to the chosen point. If they remain in coincidence throughout a complete revolution of the optical tube, the line of collimation is correct. If they do not, loosen the opposite pair of screws holding the cross-hair reticule and tighten first the one that draws the reticule toward a correct position; repeat with the other pair of opposite screws to bring the cross-hairs into position in the second direction. When finally the point and cross-hairs are coincident tighten the reticule screws, being careful not to overstrain them. The focus of the objective now falls upon the optical axis of the telescope, and at the point of intersection of the cross-hairs; i.e., the instrument is collimated. In some levels this adjustment cannot be made.

(3) To make the axis of bubble-tube parallel with the line of collimation, set up the level L at a place where two points A and B about 300 or 400 feet apart and at nearly equal distances from the instrument can conveniently be sighted. Read a rod held at each point in turn and record

Rod Reading	at A	4.527	
"	" "	B	3.213
	A - B	1.314	True Diff. in Elevation

Instrument at L_1.

Remove level to position L_2, ten feet beyond B and approximately in line with A and B. Read the rod set upon A and B as

ADJUSTMENTS OF THE DUMPY LEVEL

PRELIMINARY ADJUSTMENTS:-

Focus eyepiece on cross-hairs, and make horizontal cross-ax perpendicular to axis of rotation of instrument.

FIRST ADJUSTMENT:- Bubble

To make axis of bubble-tube perpendicular to axis of rotation, or vertical axis, of instrument.

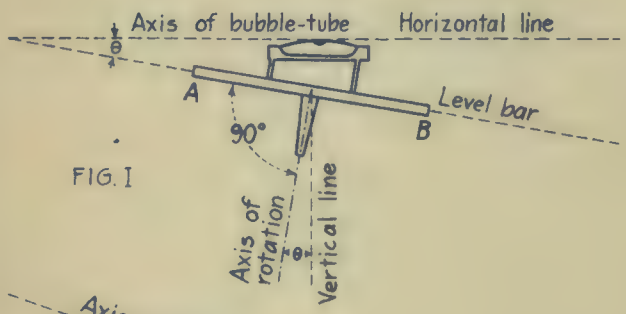


FIG. I

Level instrument carefully

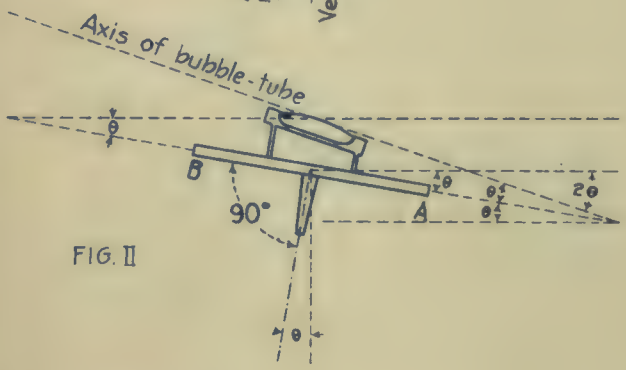


FIG. II

Turn instrument end for end on axi of rotation (this reverses bubble-tu The apparent error (2θ) is twice the true error (θ)

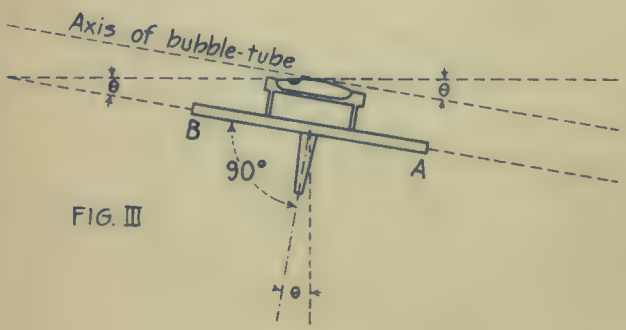


FIG. III

Bring bubble halfway back to cente by means of adjusting screws at e of bubble-tube

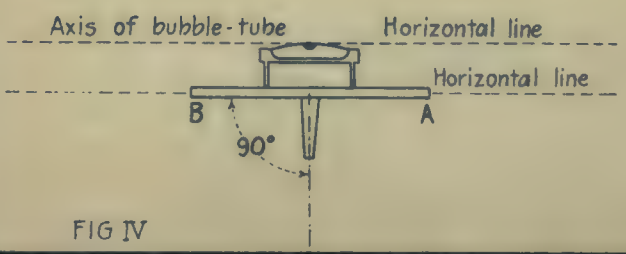


FIG IV

Center bubble by means of leveling scre

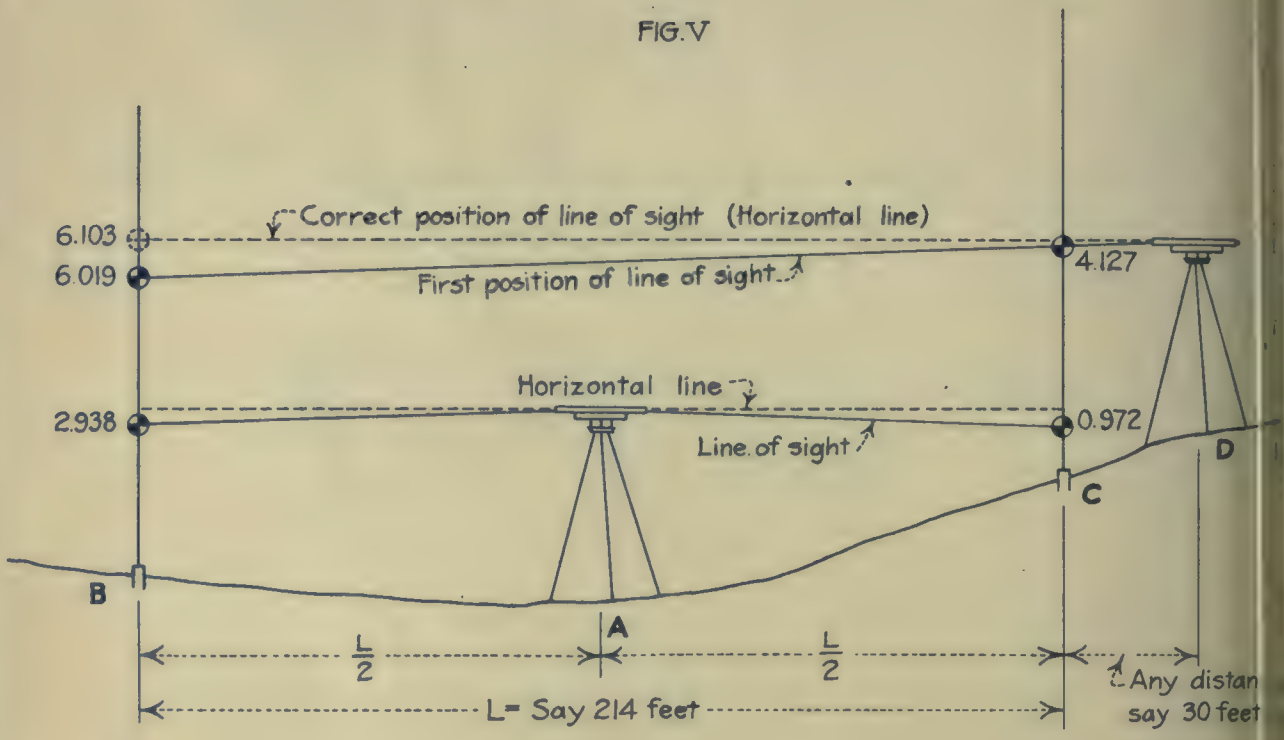
C.T. WISROC

SECOND ADJUSTMENT :- Line of sight

To make line of sight parallel to axis of bubble-tube.

Direct or "Peg Adjustment"

FIG. V



Set instrument up, in line with and half-way between two hubs, B and C, which are from 200 to 400 feet apart. Read rod on each hub. As lengths of B.S. and F.S. are equal, errors of adjustment are eliminated and the true difference in elevations of B and C is obtained by subtracting the rod readings.

Now set up again near either hub, say at D, and again take rod readings on B and C. If the instrument is in adjustment the difference in elevation of B and C, as obtained from these last two readings, will agree with the true difference in elevations. If the instrument is not in adjustment determine the amount of error.

TO CORRECT ERROR

Dumpy level :- With bubble centered, bring line of sight to correct rod reading at B by means of the capstan screws that move the cross-hair ring.

Wye level :- By means of the leveling-screws bring

line of sight to correct reading at B, then bring bubble to centre of tube by turning the capstan nuts at the end of the bubble-tube.

Form of Notes

Instrument at	Rod at	Reading	
A	B	2.938	
	C	0.972	
		1.966	True difference in elevation.
D	C	4.127	
	B	6.019	
		1.892	False difference in elevation.

This is less than the true difference in elevation, therefore the line of sight is inclined downwards. (The hub at B being lower than the hub at C)

True difference in elevation	1.966
False difference in elevation	1.892
Error in distance L	0.074 feet

Total error in line of sight in the distance DB

$$= \frac{(214 + 30)}{214} \times 0.074 = 0.084 \text{ feet}$$

Target setting at B to give horizontal line is 6.019 +0.084 = 6.103*

Note:- To determine whether the correction is to be added or subtracted draw a figure in your field book representing the specific case in hand. Show clearly relative elevations of hubs.

* For exact adjustment this rod reading should be corrected by adding the effect of curvature and refraction which, in this case is approximately 0.001 feet. The target setting at B would then be 6.104

before; we shall now have, suppose

Rod Reading at A	7.235	
" " " B	5.683	
A - B	1.552	False difference in Elevation

Instrument at L_2.

Subtract the true difference in elevation from the false difference and take $\frac{41}{40}$ of this difference.

False Difference in Elevation	1.552
True " " "	1.314
	0.238

which multiplied by $\frac{41}{40} = 0.244$. This quantity deducted from last rod-reading at A (7.235 - 0.244 = 6.991) will be the target setting for A, level at L_2, in order that the line of sight (line of collimation prolonged) may be horizontal. With the target, therefore, set at 6.991 and rod held on A, direct the telescope to the target by means of the leveling screws of the instrument, and while it is in this position raise or lower the level-tube by means of the adjusting screws at one end of the tube until the bubble is exactly in the middle of its run. It should be apparent now, that whenever the bubble is at the middle of its tube, the line of sight will be truly horizontal. The foregoing is called the "Peg Method" of adjusting the level and telescope, and should be well mastered by the student, as the same routine is followed in making this adjustment in case of the Wye-level, or the Transit.

(See A-3, Second Adjustment.)
(See Fig. A-1.)

(112) SETTING UP AND PRELIMINARY ADJUSTMENTS

These relate to instruments in general and are supplemental to the directions furnished in connection with other assignments of this course.

For purposes of field adjustment the instrument should be set up in a shaded spot on reasonably level ground, in a locality free from interference, earth shakings and local magnetic attractions. In compass adjustments avoid proximity to electric lines or pipes buried in the ground. The stretch of open space (different for different cases) should be of sufficient extent to permit all adjustments to be completed with one setting of the instrument, or, in case two or more settings are required, to permit them to be made with ease and facility, so as to secure the advantage of freedom, stability, and range.

(113) PRELIMINARY

Plant a tripod firmly, spreading the legs at angle of about 60°, so as to secure a broad base which adds to stability. Press the shoes well into the earth, which should be solid and free from loose sand or marsh. Bring the head of the instrument as nearly level as possible in the foregoing processes, so that the final work of leveling up may be done with ease and dispatch.

Examine clamp screws, pivots, etc., to see that all parts are free to turn without requiring undue force. If a telescope is a part of the instrument, uncover the objective and eye-end, and move the objective tube in to the shortest telescope length, direct

it toward the sky, away from the sun, and observe the appearance of the cross-hairs. These should stand out clear and sharply defined - even the particles of dust invariably present should be plainly visible. Sight upon some well defined distant object and observe for parallax, by shifting the eye first to one side of the eyepiece, then to the other - then up and down in similar way, and note if the cross-hairs appear to move over the field of view. If they do, parallax is present, and must be removed by careful adjustment of the eyepiece. Many telescopes have a set-screw for clamping the eyepiece when in adjustment. The eye-piece should be clamped if the instrument is to be used by one person, as in that case the eye-piece setting will not require change.

Next level up the plate or what is really the case, bring the axis of the instrument into verticality by means of the leveling screws on the tripod head. This important step should be done as follows:

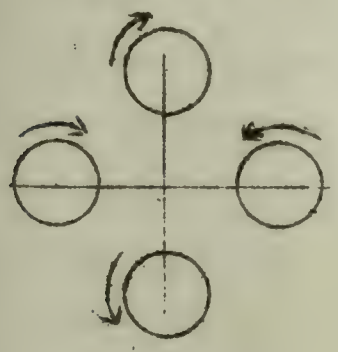


Fig. 35

Set a level in line with two diagonally opposite footscrews, as A and B in the adjoining figure.(Fig. 35) Grasp a screw between thumb and forefinger, one in each hand and proceed to turn both in or out, uniformly, and observe the direction in which the bubble moves - it will always follow the right-hand finger or the left-hand thumb - and thus bring the bubble to the mid-point

of its tube. Next set the level over the other pair of foot-screws and proceed as before. Barring possible necessary adjustment, description of which will follow these preliminaries, after a few trials the level (or levels) will be at mid-point for all positions. The level-plate will then be horizontal and the vertical axis truly vertical.

In manipulating these foot screws it may happen that they "bind", and can be moved only with increasing difficulty; this is due to one screw traveling faster than its opposite. The "binding" will be removed if you continue to turn either screw in a right-hand direction (clockwise when viewing the bottom of the screw); but the quickest method is to turn both screws together to the right (clockwise). Sometimes the binding of one pair of screws is due to the position of the other pair, and the latter should receive attention. In no event permit the screws to bind and of all things, do not continue to turn them when they do bind, except in the manner required to loosen them. All screws used in manipulating the instrument - foot-screws, clamp-screws, tangent screws, etc. - must at all times turn easily and freely. When they do not, they need attention. Do not oil screws or bearing parts. Oil will cause grit and dust to adhere to such surfaces and this in time will cause the parts to wear rapidly.

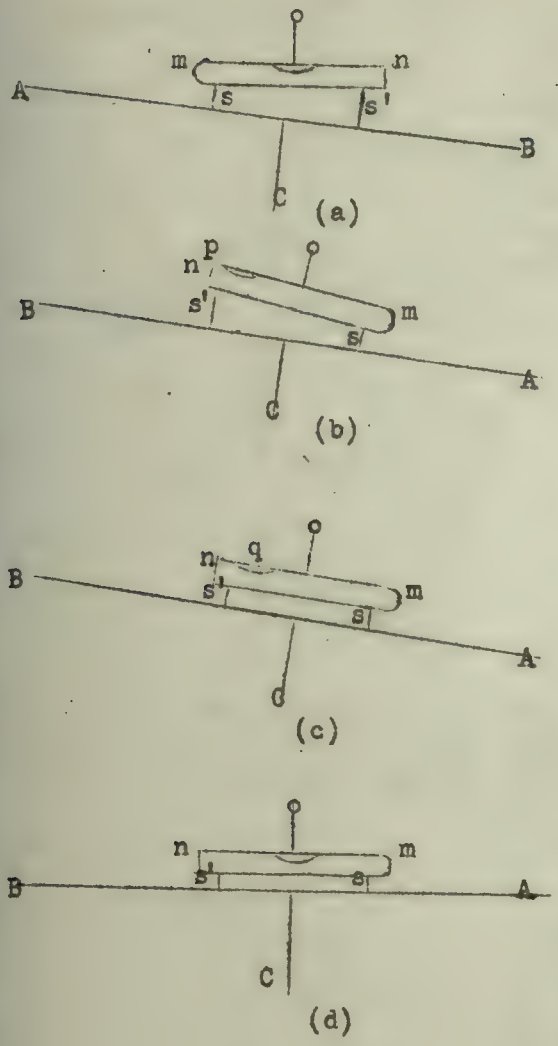


Figure 36

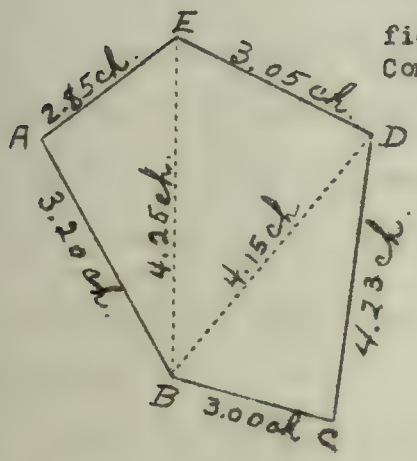
Suppose a level-tube connected by supports s s' with the line AB and it is desired to make the line truly horizontal. Adjust the line so that the bubble is at center o; reverse to position 180° as at b, Figure 36, by revolving about C. The bubble will now be at the high point p; but by shortening the support s' the tube may be brought into parallelism with AB as at c, the bubble is then at q, midway between o and p. If now the line BA is tilted until the bubble again takes the position o, as in d, BA will be parallel to the bubble-tube and in a horizontal position - the axis of rotation, C, will also be vertical (a plumb line).

REFERENCES:

Tracy	Pages 581 - 606
Raymond	" 63, 79, 108
Johnson	" 15, 63, 86, 102
Breed & Hosmer	" 25, 56, 70, 89

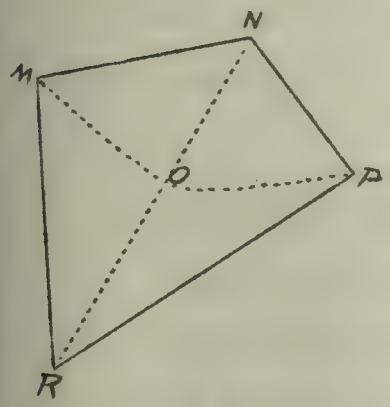
QUESTIONS:

1.



The lines of a tape survey of a field are noted on the adjoining figure. Compute the areas in acres.

2. By a single set up at O the angles about (were measured and the distances to M, N, P, & R measured; from the data thus obtained compute the area of the field



Angles		Distances	
MON	60° 15'	MO	425.0 feet
NOP	87° 30'	NO	430.5 feet
POR	92° 30'	PO	395.3 feet
ROM	117° 45'	RO	512.6 feet

ADJUSTMENT OF INSTRUMENTS (Cont.)

(114) (D) ADJUSTMENTS OF THE WYE-LEVEL

Adjustments of the Wye-level are made the same as those of the dumpy level in cases where they are of the same nature. Two adjustments of the Wye, however, are peculiar to that instrument, and must receive special treatment.

(1) To make the bubble-tube lie in the same plane with the axis of the wyes.

Raise the clips that hold the telescope tube; turn the telescope through a small angle by rotating in the wyes. If the bubble remains at its mid-point, no adjustment is needed; if not, raise or lower the end of the bubole-tube by means of the screws that control its lateral motion, until the bubble returns to its mid-position. It may require two or more trials to do this. (See A-4, First Adjustment.)

(2) To make the axis of the wyes perpendicular to the vertical axis.

Level up the instrument and turn 180° upon the vertical axis; bring the bubble one half way back to center by means of leveling-screws and adjust to center by raising or lowering one of the wyes by means of the capstan screws supporting the wye. Repeat the adjustment to insure accuracy. (See A-5, Third Adjustment.)

(115) (E) ADJUSTMENTS OF THE TRANSIT

A complete transit, that is, one having a bubble-tube on the

ADJUSTMENTS OF THE WYE LEVEL

PRELIMINARY ADJUSTMENTS :-
Same as for the dumpy level.

FIRST ADJUSTMENT :- Line of sight.
To make line of sight coincide with the axis of the collars

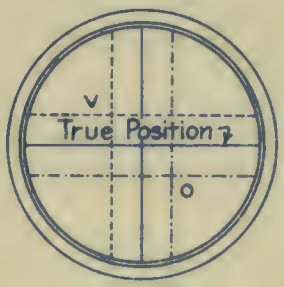


FIG. VI

Without leveling set cross-hairs on a distant point "o". Revolve telescope half-way around in the wyes, (not end for end) intersection now appears at "v".

TO CORRECT ERROR By means of the capstan screws that move the cross-hair ring bring the intersection of the cross-hairs to a point mid-way between "v" and "o".

When this adjustment is accomplished the intersection of the cross-hairs will remain fixed on one point throughout a complete rotation of the telescope.

SECOND ADJUSTMENT :- Bubble
(a) To make the axis of the bubble-tube parallel to the bottom of the wyes.

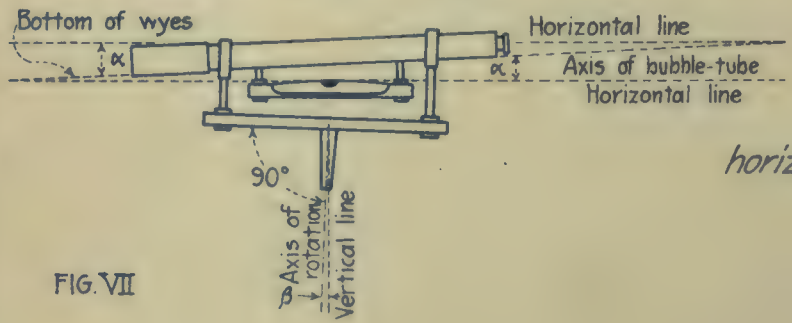


FIG. VII

Level up and clamp horizontal motion.

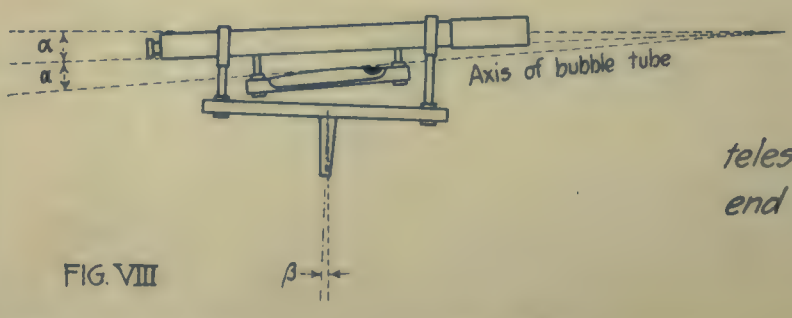


FIG. VIII

Unfasten clips, remove telescope from the wyes, turn end for end, and replace in wyes

C.T. WISKI

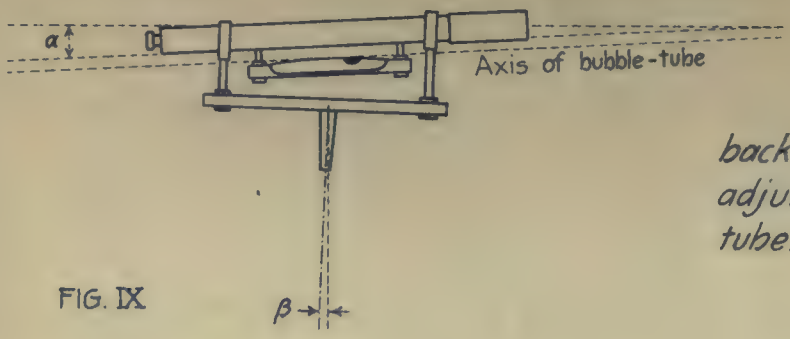


FIG. IX

Bring bubble half-way back to center by means of adjusting screws at end of bub tube.

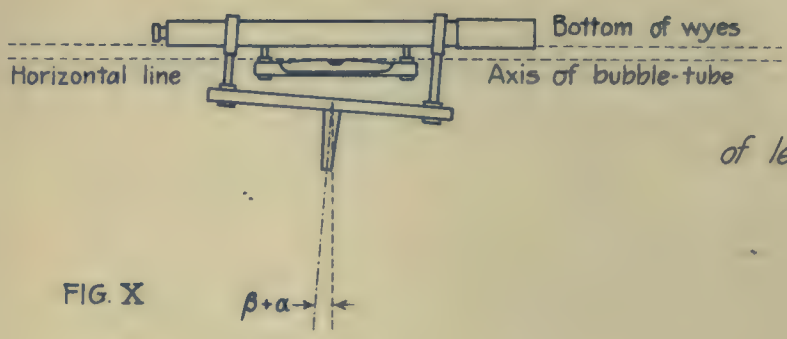


FIG. X

Center bubble by mea of leveling screws.

(b) To make the axis of the bubble-tube lie in the same plane with the axis of the collars.

Level up and clamp horizontal motion. Loosen clips, then swing bubble-tube through small angle by revolving telescope If the bubble moves away from the center it should be adjusted

TO CORRECT ERROR Bring the bubble all the way back the center by means of the capstan screws which control the lateral motion of the bubble-tube.

(c) Repeat bubble adjustment (a).

Note:- If the collars are the same size these adjustm will make the axis of the bubble-tube parallel to the line of s This is an indirect method of making the adjustment. For a di method use the "peg adjustment".

THIRD ADJUSTMENT:- Wyes.

To make the axis of the bubble-tube perpendicular the axis of rotation, or vertical axis of instrument.

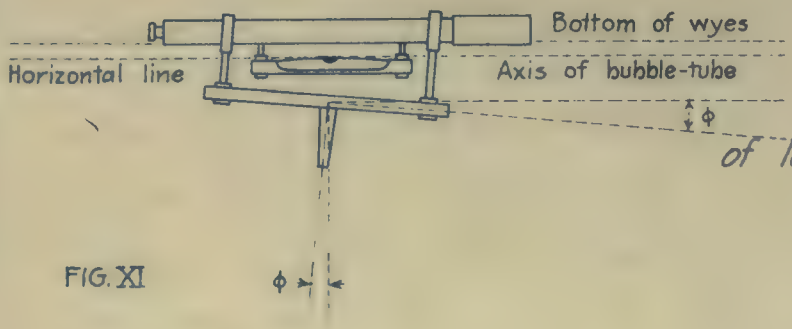


FIG. XI

Center bubble by me
of leveling screws.

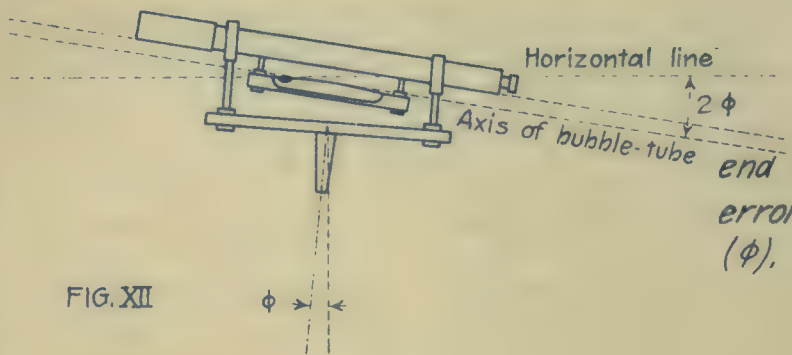


FIG. XII

Turn instrument end
end on axis of rotation. Appa
error (2ϕ) is twice the true er
(ϕ).

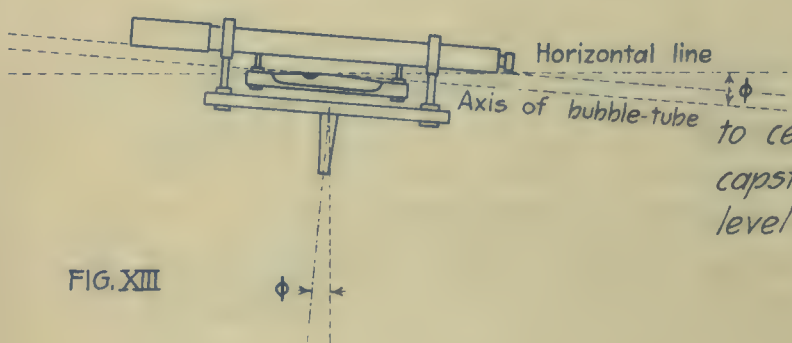


FIG. XIII

Bring bubble half-way
to center by means of the lar
capstan nuts at the ends of
level bar.

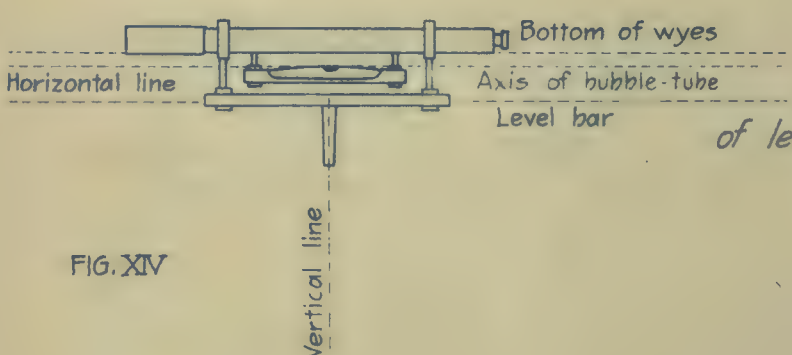


FIG. XIV

Center bubble by me
of leveling screws.

Vertical line

telescope and a vertical arc in addition to the usual plain instrument (Plate III, Assignment IX) is preferred for the work described here, but the surveyor should learn to adjust "any old" instrument that he may be called upon to use, for all adjustments become more or less deranged by the action of the elements, etc., or by rough or careless usage, or through accident.

Preliminary:

Set up the transit in an open space, about 200 to 400 feet long, on as nearly level ground as possible, preferably in the shade. Place the instrument over a hub or stake set in the ground by means of the plumb-bob; center it over a definite point in the hub. This done, it is unnecessary to select any other initial point throughout many of the adjustments that follow; if the instrument is removed to any other setting during a given adjustment, this new setting should then be sufficiently identified for occupancy at any later time.

(1) To make the plate bubble-tubes perpendicular to the vertical axis of the instrument:

Put one of the level-tubes parallel to a pair of diagonally opposite foot-screws; this brings the other tube parallel to the remaining pair of screws. By manipulating the foot-screws as directed for the leveling instrument, Assignment XIII, Art. 113, bring each bubble to the middle of its tube; turn the plate on the lower spindle through 180°, which will reverse both bubbles over their respective set of foot-screws, and note the position of each

bubble in turn. If they are at mid-point for each position, no adjustment is necessary; if not, bring either (or both) bubbles one-half way back to center by means of the foot-screws and then exactly to center by means of the screws that hold the level-tube to the plate. Repeat the whole process for test and subsequent adjustment - one repetition is not always enough.

Some of the adjustments to follow do not require this bubble-tube-on-plate adjustment, but any other adjustment is made with greater facility and assurance when this is done. This secures a normal position of the transit (i.e. vertical axis truly vertical), which is always desirable. In some transits one of the bubble-tubes just spoken of as situated on the horizontal plate, is not so situated, but is attached to a standard that supports the horizontal axis (the trunnions) of the telescope; the principle is the same, however, and the elevated position of this level-tube in no way affects the result.

(2) To make the upright cross-hair vertical:

Direct the line of sight to some sharply defined point at about 50 ft. from the instrument; focus one end of the upright hair exactly upon it; and then, by turning the telescope tube on the trunnions, note whether the point "tracks" upon the cross-hair. If it does, no adjustment is needed; if not, loosen all four screws that hold the cross-hair reticule, and turn it around by gently tapping the screws. Reset the screws, but be very careful not to overstrain any of them in so doing. In the absence of a convenient

point for sighting upon, a plumb-line may be suspended and used instead. Although this plumb-line requires more time and trouble, it is sometimes preferred. If the horizontal cross-hair is placed (as it should be by the maker) at right angles to the upright one, the horizontal hair is now in adjustment also. However, it is more important that the vertical cross-hair be truly vertical; hence the reason for testing and correcting this one primarily.

(3) To make the line of sight lie in a plane at right angles to the horizontal axis of the telescope:

This requires two adjustments - (a) When the line of collimation of the telescope is horizontal; (b) when the line of collimation is in any other position in altitude.

(a) With transit set over a fixed hub P as described in (1) direct the line of sight toward a point A about 200 ft. distant and at about the same elevation as that of the telescope (H.I. of transit). Turn the alidade plate 180° about the vertical axis and set another point B at about the same distance as for A and at H.I. Now invert the telescope (plunge it) and sight again upon A. If A is bisected by the line of sight, the adjustment (a) is correct; if not, clamp the instrument and set a marker A' in the line of sight near A. Take 1/2 of the distance AA', and set if off from A toward A'; call this point D. With the plates and the telescope both clamped in position, loosen the right and left screws that hold the cross-hair reticule, and tighten the one that will bring the vertical cross-hair over the point D.

Note: To test this adjustment, three points in line through P, at H.I., will be on a straight line. Every position of the telescope when bisecting one point will, by reversing or plunging (inverting) also bisect the other point.

(b) Set up the transit in a locality where observation may be made on some high point, A, as the top of a tall building, a spire, or a flag-pole, and carefully level the plate. As this adjustment is to correct for the intersection of two vertical planes the point aloft should be high, but not at a great horizontal distance from the instrument. Sight first upon the high point; clamp the plates while bisecting the point, turn the telescope on its horizontal axis (in the trunnions), and sight a marker placed at a convenient distance, if possible directly below A and at about the elevation of the center of the instrument (H.I.). Call this point on the ground B. Reverse on either spindle, plunge the telescope and again bisect A, the point aloft. Turn the telescope down on the trunnions and direct toward B; if B is bisected by the line of sight the horizontal axis is in adjustment; if B is not in the line of sight place another marker B^1, in line near B. The adjustment is then made by raising or lowering the movable end or trunnion of the horizontal axis, by means of the adjusting screws in cap at movable end, until the line of sight bisects the point midway between B and B^1. Go through this whole process a second time for test and subsequent adjustment.

(4) To adjust the Level on Telescope.

This may be done by the Peg Method described in connection with the Dumpy Level, used likewise with the Wye level. (See Assignment XIII) A simple method of making this adjustment is the following: Set two stakes A and B at the same level. This can be done by choosing a nearly level stretch and carefully setting the stakes by means of the transit telescope, or better, by means of an engineer's level. Or if a sheet of water of considerable extent may be near at hand this may be used for determining a level line by driving the stakes so that each projects one or several inches above the surface and at the same level. Now with the transit set up and most carefully leveled near A, sight on a target set at H.I. on B; adjust the level on telescope by means of the screws that hold it to the telescope, bringing the bubble to the center. For test and subsequent adjustment set up at B and observe on target set at H.I. on A. (Here H.I. means height of the line of collimation at intersection of vertical axis, i.e., the center of the instrument.)

(5) To adjust the Vernier Index of the Vertical Arc.

With the telescope bubble exactly in adjustment and at the center of its run, and with plate levels in perfect adjustment, loosen the screws that hold the vernier of the vertical arc (or circle) and move until the 0 or index ($\wedge$) coincides exactly with the proper limb division, 0 or 180°. If there are two opposite verniers, both should be adjusted to read exactly 0 and 180° when both the telescope and its level are horizontal and in adjustment.

This adjustment is important and most convenient, but by no means essential, as the vernier error may be observed without moving it into correct adjustment and applying this as correction to readings of vertical angle for every case, adding or subtracting the correction as circumstances may require. Some transits have a small level attached to the vernier of the vertical arc, which obviates the adjustment between telescope level and arc, as this level and arc adjustment is difficult to make and therefore troublesome.

(6) Adjustment of the Gradienter.

This is made when the preceding adjustments are made. Loosen the swinging arm (a sort of lever) that is actuated by the gradienter screw, which is likewise the tangent screw to the horizontal axis. Move the milled head and knife gauge both to zero and then tighten the arm upon the trunnion. Be sure that the settings of any of the transit parts concerned in this adjustment are not disturbed in making this adjustment.

116) GENERAL OBSERVATION ON ADJUSTING

Besides remembering that the adjustment of instruments is of great importance, you should give attention to other things necessary in sound practice.

First. The adjustments just described for the transit should be gone over in the order they are given, unless it is obvious that some of them are correct and hence do not require attention. Some adjustments can not be made until certain ones precedent

thereto have been made; e.g., adjustment 3b must follow 3a to accomplish the correction, and therefore 3a should at least be tested out.

Second. The order of procedure in any particular adjustment will be (1) the desired relation; (2) the test; (3) the method of adjusting; (4) a final test, usually made with greater care, which will prove of immense satisfaction to the careful engineer.

Third. In making any alteration by loosening, tightening, or shifting screws or other parts, do all in a workmanlike manner. A slotted screw should be turned with a screw-driver that fits the slot, a capstan screw with a pin that fits the hole in it, milled-head screws with the thumb and finger, not with pliers or a wrench. Never leave screws loosened where they should be firm, nor overstrain them in tightening them.

Fourth. When a general overhauling of an instrument is required, do not attempt to bring any given part into perfect adjustment by a "once over", but proceed by making the series of corrections as accurately as possible in the order given above. When the whole series has been followed out, return to number one and repeat each one for test. Of course if test does not reveal that the error has been corrected, repeat the process necessary to accomplish this.

(117) (F) ADJUSTMENTS OF THE SOLAR ATTACHMENT (Saegmuller). (See Plate VIII, Assignment XII.)

These adjustments must be made after attaching the solar upon the transit telescope.

Preliminary.

See that the screw that connects the solar is turned down to a firm position. Bring the levels of the alidade plate to center and also the level on transit telescope; set the index of the vertical circle.

(1) To make tha axis of the bubble-tube in the solar telescope parallel to the axis of the same. This is accomplished exactly as for the transit telescope by the method of reversion.

(2) To make the polar axis of the solar in line with the vertical axis of the transit.

Be sure that the transit's vertical axis is truly vertical, then bring the bubble of the solar telescope to center, and test for all positions through 360°. If not in adjustment, employ the method of reversion.

(3) To make the lines of collimation in the two telescopes parallel.

With the preceding adjustments fully made, sight both telescopes upon a vertical line (a suspended plumb line or corner of a building) at 100 feet or more away. Clamp the telescopes in this position and carefully level the bubble on transit telescope so as to bring the lines of sight of both telescopes into the same plane. Now measure the distance between the axes of the telescopes along the polar axis. Draw two distinct parallel lines on a piece of paper, or better, of cardboard, at this same distance apart. Fasten up the card with the lines horizontal at 100 feet or more away, and at the same level, and direct the telescopes toward it, aligning

the transit cross-hair upon the lower line. The solar cross-hair should then be on the upper line when its bubble is at center; if not, alter the position of the cross-hair reticule in the solar telescope until this is accomplished. Test by revolving the whole instrument around upon the spindle of the lower plate of transit. The bubbles of the alidade-plate, the bubble of the transit telescope, and also that of the solar telescope should remain at center for all positions through 360°. The lines of sight of both telescopes should intercept the same interval (that between their axes) at all distances. All seven adjustments, two of the solar and five of the transit, must be made with scrupulous care to secure satisfactory results when using the solar.

(118) (G) ADJUSTMENTS OF THE PLANE-TABLE

Plane-tables embody the same parts as the transit. The leveling head corresponds to the similar part in the transit; the board or planchette to that of the lower plate; the ruler, surmounted with sights or telescope, to the alidade of the transit. The levels are variously attached; some have only one level upon the ruler, others have two levels at right angles so attached, while still others have no level attached to the ruler, but are furnished with a bar-level that may be placed at any desired position on the board, for the purpose of adjusting the table to a horizontal position.

(1) The adjustment of the leveling device in any case is accomplished by the method of reversion, the levels after being

turned through 180° are brought half way to center, first by the leveling head, then by the level adjusting screws. In case of the bar-level the adjustment may be made in a manner similar to that described for the carpenter's level in Assignment XIII.

(2) To make the vertical circle (arc) read zero when the telescope bubble is at center.

Carefully level the telescope and set the vernier index at zero of the arc.

(3) To make the edge of the ruler a true fiducial line.

Upon a smooth surface draw a fine pencil line the full length of the ruler; reverse the ruler end for end setting it against the line and note whether the line and the ruler edge again coincide. If not, the edge should be straightened. This requires skill and necessary tools; the instrument maker is the best one to do this. If, as might happen, one half of the ruler's edge curves in one direction and the other half in the opposite direction (i.e. half is concave, half convex), a reliable test is to move each half along the line to the opposite part; the deviation in either portion will then immediately reveal itself.

(119) In conclusion on the subject of adjustments a few generalizations are here presented.

Be sure that any adjustment is required before making it, and that the method is fully understood before attempting any alterations, and then use the proper means and proper tools. A

novice should always work under the advice and direction of someone having the knowledge and skill necessary in any case.

Needless taking of instruments apart, cutting, filing or scraping parts to fit or attempt to repair and adjust them must never be done except in extreme cases. Generally send the instrument to the maker.

Good work can often be done with instruments out of adjustment, provided the user knows how to determine certain relations. For example, the plate levels need not be in adjustment provided one knows what position their bubbles should have when the vertical axis is truly vertical. For the horizontal plate is horizontal if the plate bubbles remain in the same positions in their tubes throughout a complete revolution. Again, the line of sight need not be parallel to the axis of the level on telescope provided the index of the vernier of the vertical arc (circle) is at zero when the line of sight is horizontal; the converse condition is also permissible provided one knows and applies the index error of the vertical circle.

Again, in running levels with an engineer's level or transit, it is enough that the level (attached to the telescope in either instrument) be at the middle of its run, or any other definite position, when taking sights at <u>equal distances</u> from the instrument.

But the liability to error is too great, and corrections are too easily neglected in all such cases, and it is, therefore, always

best to have instruments fully and nicely adjusted. It is in such conditions that work can be done easily, rapidly, and with the satisfactory assurance that the liability to instrumental error has been mostly eliminated.

Among the problems set for student work several will be given in the work of adjusting.

REFERENCES:

Tracy pp. 581-606
Raymond pp. 63, 79, 108
Johnson pp. 15, 63, 86, 102
Breed, H., pp. 25, 56, 70, 89, Vol. I

QUESTIONS:

1. The needle of a compass is bent. How may the error in bearing from this cause be eliminated?

2. In compass use which of the two plate levels is more important and why? (Give reasons)

3. How may a temporary setting of the plates of a transit in a horizontal position be secured without adjusting the plate levels?

4. If a transit has a full vertical circle, how may a level line be determined without the aid of the level on telescope?

5. How may a level line be determined by use of a transit in the absence of a vertical circle and a level on telescope?

UNIVERSITY OF CALIFORNIA EXTENSION DIVISION
CORRESPONDENCE COURSES IN ENGINEERING SUBJECTS

Course 1A Elements of Surveying Swafford

Assignment 15

METHODS OF LAND SURVEYING

FOREWORD

This assignment will set forth the several methods of Land Surveying, explain the purposes of such surveys, and give details of the procedure in each case.

(120) PURPOSES OF LAND SURVEYING

(a) To lay out the bounding lines and locate their intersection; to fix necessary points so that they may be identified and re-located by reference to certain known or established lines or points; to make a record in the form of notes of data used in the survey; and to map such a survey in suitable manner.

(b) To retrace the lines and the angles of any survey, and to re-establish any or all points in this survey for the purpose of identifying the various features of a tract or parcel of land.

(c) To measure established lines and angles of any tract for the purpose of determining the content or area of such tract.

(d) To run lines in extent and direction for the purposes of sub-dividing or "parting off" of lands by parcels either in form or content.

Land surveys may be executed either by use of the compass and chain, as in farm tracts; or by transit and tape in cases requiring more refined measurements, as in city surveying. (See Assignment XXVI on City Surveying.)

(121) Since in most cases the tract of land to be laid out or surveyed forms part of the U. S. Public Land Survey, it is necessary to follow established lines of such survey. This requires a knowledge of the methods and practice of the Public Land Surveys, which will be treated in Assignments XIX and XX of this course. The work of the surveyor in such cases is to reproduce the lines and points as established by the Public Land Survey. This becomes a simple matter of identifying certain points previously set.

As an example of this, suppose that a certain tract is described as the N. E. Quarter of Section 23, Township 2 North, Range 3 East, Mt. Diablo Meridian and Base. Here is clearly a case of identifying township and section corners. The number of the Township (2 North) and Range (3 East) are the first important designations and these constitute the co-ordinates necessary for locating the region of this particular tract of land which is further described as being the Northeast portion (1/4 square mile) of Section Number 23.

The nearest Township corner is the Southeast corner of T2 N, R3 E of Mt. Diablo Base Line Monument and is north 6 miles and east 18 miles from said B.L. monument. Thence to the southeast corner of Section 23 is two miles north by one mile west where the southeast corner of Section 23 should be found. Then, by measuring 80 chains due north, the northeast corner of section 23 is also identified. If the point 40 chains north from the southeast corner was located in the running of the line northward, it

may now be checked by measuring southward from the northeast corner and a proper monument established. Also the northwest corner must be determined by measuring westward from the northeast corner of Section 23 a distance of 40 chains and checking by measuring the full 80 chains (1 mile) to the northwest corner of Section 23. A monument is set at the 40 chain point. Thence measure 40 chains south and check by measuring 40 chains east to the first 1/4 section division established on the east line of Section 23. All this is necessary since the Public Land Survey sets monuments only at section corners and at quarter-section points on boundaries of sections. All interior quarter section corners must be located on the land by the local surveyor.

A reference to the shaded quarter section in Section 23 of the adjoining diagram of a township will further elucidate the foregoing description and procedure.

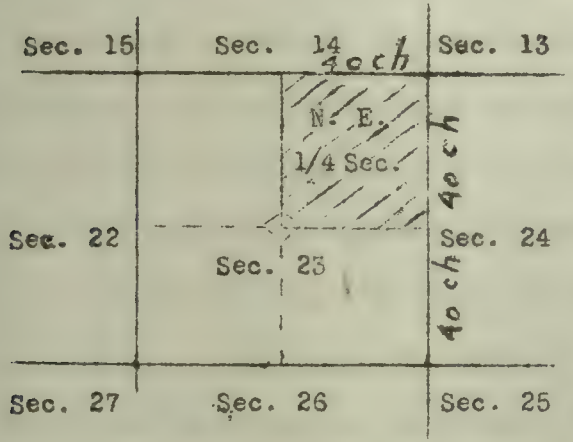


Figure 37

The function of the U. S. surveyor was completed with the establishment by him of the section corner monuments; the local surveyor using these monuments sets the quarter-section corners for his client. A knowledge of the methods and practice of Surveys of the Public Land is essential to the surveyor who

would do work worthy of an expert and faithfully serve those employing him. As a large variety of problems of the nature of the foregoing present themselves for solution, you are advised to acquaint yourself with the instructions furnished by the General Land Office of the U. S. as no specific instructions are furnished from any other source.

Deeds to lands give descriptions from which the surveyor is required to retrace or lay out the property lines showing the position of corners and giving the angles or bearings of lines. These descriptions usually name the parcel or lot of land and often give "metes and bounds" which are the lengths and directions of lines bounding the tract. From the data so supplied the surveyor goes into the field, sets up the instrument at a corner of the property that is distinctly marked (or perhaps most convenient) and proceeds to measure angles and courses as set forth in the description. It is his function to reproduce to the best of his ability the points and lines there given and to mark by proper stakes of other means these points upon the land. At least one prominent corner should be tied in. That is, two or more points, the bearing and distance of which are carefully measured, are chosen and described in order that the point so tied may be readily relocated, should the stake or monument marking it be removed.

A farm or city lot which is to be fenced or on which buildings or other improvements are to be erected should be carefully surveyed before beginning such improvements. The location of the

corners, fence lines and sites of buildings, ditches, drainage lines, etc., can consistently form a part of such survey, and should be executed with a precision in accordance with the importance of the work. Carelessness in this regard has often been the cause of serious and expensive mistakes and has led to trouble and litigation, which a correct survey would have prevented. This is especially true of divisional lines in both farm and city property, especially in the latter where buildings are frequently made to follow property lines.

In case a survey is to be made to relocate the points and lines of an old survey where the bearings are given with relation to the magnetic meridian, it is essential that the former declination of the magnetic needle be shown. Should the declination at time of the original survey be a part of the record (which should be noted on the original notes, but unfortunately is not always so) and should the present declination also be determined, this becomes a simple matter. But in the absence of such data they must faithfully be recomputed for proper guidance in the survey. Assignment VI on Compass Surveying gives adequate instruction for this purpose, but at least one course should be rerun on the ground for the purpose of fixing such course with respect to the resurvey and of properly orienting the property. Any variation of the declination should be correctly applied to every course in the resurveys.

The surveyor has no warrant for establishing points or running lines otherwise than as given in the original. This may not

apply in case of a monument or stake that may be lost or obliterated, in which case the surveyor relocates and reports the restoration of such monument to the best of his ability and in conformity with the remaining features of the survey in which the locations are known and distinct. (See Assignment XL on the Judicial Functions of the Surveyor.)

Each survey for location or resurvey should embody in the report a full and complete set of notes and in most cases a map of the tract, showing tie lines, witness marks, courses in bearing and distance, interior angles, and any explanatory remarks that may render the notes clear and intelligible to others. A clearly executed sketch is usually made a part of the notes and should not be omitted when needed to clarify otherwise obscure or ambiguous data. A little care in this matter often adds to the value of the report and obviates the necessity of much labor otherwise.

The survey of a tract for the purpose of finding the content or area constitutes a third purpose and consists generally in re-running the courses, measuring the distances and checking the points upon the tract of land. If the survey is for area only, it is not essential that attention be given to the orientation or even to the declination provided the corners of the tract can otherwise be readily located. In such case the bearings and distances as related to the form of the tract itself are sufficient. However, it is most expedient that the survey for area be conducted

in the usual manner by traversing with compass and chain, or, where special accuracy is required, with transit and tape.

Bearings may be either magnetic or true bearings. Where the latter are to be taken with the compass, it is convenient to set off the declination with the variation arc and to take such bearings directly in the field. These may be computed, however, from recorded magnetic bearings in lieu of setting off the variation.

The surveyor should, if practicable, traverse the entire tract, set up at each corner, take the back and forward bearings, and check by measuring interior angles. He should measure all sides of the field. If it contains any meander lines such as those of large streams, ponds, or lakes, a set of off-sets from a measured straight line to the meander should be measured and charted. In measuring across streams and bodies of water or across ravines or past any unavoidable obstruction sufficient data should be taken in the field to enable a correct computation to be made subsequently. Hence, check up all such data taken in the field before leaving it and be assured that such are full and correct. This will simplify the office work.

Upon completion of the survey the error of closure should be noted. This may be determined by measuring the distance from the end of the last course as taken in the field to the point of beginning of the first course and dividing this small distance by the measured perimeter of the field. If this error of closure exceeds 1 in 500, the measurements, especially of doubtful courses,

should be repeated for check or correction. In case of transit surveys, where great accuracy is required, this error of closure ought not to exceed 1 in 5000 or perhaps 1 in 10,000. The error of closure depends upon both the angle measurement and the line measurement, and when both are made with equal care, the error should be distributed accordingly. Check lines, such as diagonals, are often of value in locating the presence of error and where time or expense are not too great these lines should be measured. Too great pains cannot be taken where precision is required;

(122) Two other methods of survey for area (a) measurement of sides with a sufficient number of diagonals to divide the figure into geometric figures convenient for computing their areas. Most all fields may be divided into triangles by a suitable number of diagonals joining non-consecutive vertices. In such case the trigonometric formula - Area (of each triangle) = $\sqrt{S(S-a)(S-b)(S-c)}$, where a, b, c, are the lengths of the sides and S = 1/2 (a+b+c). Frequently the computation is simplified when a rectangular or trapezoidal portion may be measured off.

If any portion be bounded by irregular curved lines as in case of the meander of a stream or body of water, etc., suitable off-sets should be made from a straight diagonal line and the area computed and added to the remainder of the field. Assignment XVII will consider in more detail the mathematical methods of computing areas.

(123) The remaining method of surveying for area (b), by a single set-up of the instrument, may now be considered.

Select any convenient point within the field or at one corner, if suitable triangles can be laid out from such setting; measure the angles (bearings) formed by lines radiating from the point occupied to the corner of the field; also measure the lines including these angles and by trigonometry compute the areas of all the triangles formed by the sides of the traverse and the radiating lines. The total angle about a central set-up will of course be 360° and is a convenient check for the angle measurement, but is by no means absolute; the discrepancy should be small. If the set-up is made at one corner then the interior angle at this corner should be measured as a check.

The trigonometric formula for computing area in this case is Area = 1/2 the product of the including sides and the sine of the included angle. Example: Given in the triangle $\overset{\wedge}{OAB}$ the sides OA, OB, and the angle O; then area =

$$1/2\ \overline{OA} \times \overline{OB} \times \sin O.$$

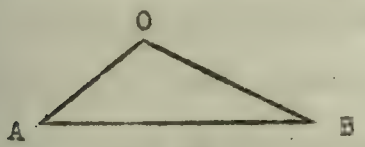


Fig. 38

When employing the method of a single set-up it is most convenient to take stadia measurements of the radiating lines, which are usually sufficiently accurate for the uses of such surveys.

These last two methods (a) with tape only, and (b) by a single set-up are quite suitable for obtaining the areas covered by crops, meadow lands, etc., and are especially applicable to measurements

of such lands when devastated by fire, or for estimating the extent of damage for insurance companies, railroads, and others interested in loss adjustment.

(124) The subject of subdivision and parting-off of land will be specially treated in a subsequent assignment (XVIII) but a few general directions are given here.

Before proceeding to a subdivision it is essential that a full and correct survey and map of the entire tract be made, if such is not already available. This survey should show the location of all corners with a suitable number of tie-lines, courses of boundaries, meanders, roads, buildings, and other improvements.

In the office the subdivisions should be determined by measurement or computation and if expedient should be clearly charted upon the map. The actual subdivisional survey should be the locating of these protracted points and lines upon the land, marking the same with convenient monuments or stakes of a more or less permanent character. If portions are to be set aside for public or joint communal use, such as roads, lanes, streets, or parks, the proper designations must be shown upon the map and staked out in the field.

It is sometimes required to subdivide into aliquot parts - halves, thirds, fifths, etc. in the distribution of estates as devised by will or otherwise, or to part-off such proportional parts as may be required. Here the problem becomes one of a mathematical nature. As these problems are purely geometric or at

most trigonometric in their scope, it is unnecessary to deal with them here.

When the divisional line is to be run perpendicular to a given side, as a line of road or street, or parallel to a given side, or otherwise in direction, the "falling" of the line and its direction must be first determined and such lines as are required to accomplish this are then accurately staked off in the field. (See Assignment XVIII on "parting off" of land.)

References:

Breed & Hosmer, Vol. I, Chap. V.

Tracy, Chap. XII.

Raymond, Chap. VII.

Johnson, Chap. VII.

QUESTIONS:

1. Explain how a line maybe prolonged where an obstruction, such as a building, stands upon a line.

2. How may a line be prolonged across a river or other body of water?

3. Show how a parcel of land may be surveyed by traversing when the corners cannot be occupied owing to fences or buildings.

4. Explain the use of random lines in surveying. What do you understand by the slope or off-set of such a line?

UNIVERSITY OF CALIFORNIA EXTENSION DIVISION
CORRESPONDENCE COURSES IN ENGINEERING SUBJECTS

Course 1A Elements of Surveying

Assignment 16

LATITUDES AND DEPARTURES CO-ORDINATES

FOREWORD:

The positions of points in a plane are often conveniently expressed by the distances from certain assumed axes, which are the co-ordinates - to the right or left, abcissae; above or below, ordinates. Since the bearings of lines are chosen with respect to a meridian (either true or magnetic) the common designation of rectangular co-ordinates is conveniently made the distance north or south on the meridian, called latitude and the distance east or west from the meridian, called departure. The present assignment will explain the manner of so representing courses in surveying and the application of co-ordinates to Land Surveying.

(125) The course AB in Figure 39 is defined by giving the the angle of bearing θ and the distance AB. The latitude (distance north on the meridian) Am, is found by multiplying AB (course distance) by the cosine of θ. The departure (distance east from the meridian) An, is found by multiplying AB (course distance) by the sine of θ.

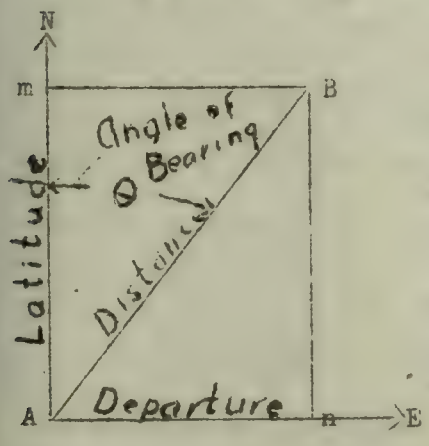


Fig. 39

Here the bearing of course Ab is northeast, or we say it lies in the N.E.

quadrant; in this case the latitude is a north latitude and the departure is an east departure. Had the bearing been N8°W, then the latitude would also be a north latitude, but the departure would be a west departure.

So also in the S.E. quadrant the latitude is south, the departure east; in the S.W. quadrant the latitude is south, the departure west.

For convenience latitudes north are plus, latitudes south, minus; departures east are plus, departures west, minus. When entering into computations these signs are always prefixed.

North latitude and south latitude are also called northings and southings respectively; east and west departures are known as eastings and westings, respectively.

The latitude then is the product of the distance into the cosine of the bearing; and if north, it is positive (+), if south, it is negative (-).

Lat. = Dist. x cos Bearing.

The departure is likewise the product of the distance into the sine of the bearing; and, if east, it is positive (+); if west, it is negative (-).

Dep. = Dist. x sin Bearing.

In Figure 40, the angles by bearings and the latitudes and departures are graphically represented.

The course AB has N. Lat. = 9, E. Dep. = 6; these may also be taken as the lines Bm and Am respectively.

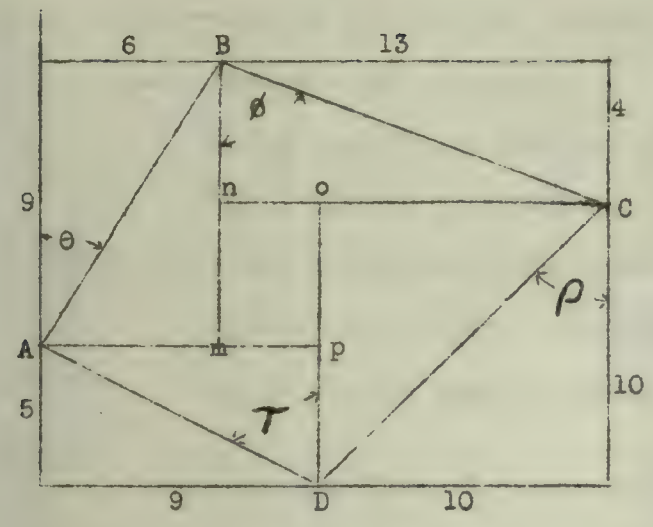


Figure 40

Distances and Bearings of Courses

Course	Bearing	Distance
A - B	N θ° E	$9^2 + 6^2$
B - C	S ∅ E	$13^2 + 4^2$
C - D	S ρ W	$10^2 + 10^2$
D - A	N τ W	$9^2 + 5^2$

Calculation of Distances

A - B = $\sqrt{117}$ = 10.82

B - C = $\sqrt{185}$ = 13.60

C - D = $\sqrt{200}$ = 14.14

D - A = $\sqrt{106}$ = 10.30

Calculation of Angles of Bearing

θ = 33°41!5 Tan θ = 6/9 = 0.6667
∅ = 18 00.3 " ∅ = 13/4 =3.2500
ρ = 45 00.0 " ρ = 10/10 = 1.0000
τ = 38 00.6 " τ = 9/5 = 1.8000

Natural Functions

Tabulation of Results

Course	Bearing	Distance	Computed N.Lat.	Computed S.Lat.	Computed E.Dep.	Computed W.Dep.	Constructed Lat.	Constructed Dep.
A - B	33°41!5	10.82 ch.	9.00		6.00		+9.0	+6.0
B - C	72 53.8	13.60 "		3.99	13.00		-4.0	+13.0
C - D	45 00.0	14.14 "		10.00		10.00	-10.0	-10.0
D - A	60 56.7	10.30 "	5.00			9.00	+5.0	-9.0

	A - B	B - C	C - D	D - A	
Lat.	9.002	3.989	9.999	5.002	Lat.
log Lat.	0.95437	0.60079	0.99994	0.69916	log Lat.
log cos	9.92014	9.46725	9.84949	9.68632	log cos
log Dist.	1.03423	1.13354	1.15045	1.01284	log Dist.
log sin	9.74407	9.98035	9.84949	9.94159	log sin
log Dep.	0.77830	1.11389	0.99994	0.95433	log Dep.
Dep.	6.002	12.999	9.999	9.002	
	A - B	B - C	C - D	D - A	

The computation of latitudes and departures from the course (given angle of bearing and distance) is much shortened by logarithmic method. The formulae

Lat. = Dist. x cos Bearing, and

Dep. = Dist. x sin Bearing,

are conveniently put in the logarithmic form as follows:

log Lat. = log Dist. + log cos Bearing, and

log Dep. = log Dist. + log sin Bearing.

Example:

Given bearing N23°45'E, distance 463.7 ft. To compute the Latitude and Departure. Arrange in the following convenient form:

	Lat. = 424.43	(a)	Add (c) and (d) which give log Lat.,
log	Lat. = 2.62781	(b)	
log cos	Bear. = 9.96157	(c)	then the anti-log is 424.43, (a).
log	Dist. = 2.66624	(d)	
log sin	Bear. = 9.60503	(e)	Again add (d) and (e); this gives log Dep.,
log	Dep. = 2.27127	(f)	
	Dep. = 186.75	(g)	(f), and the anti-log is 186.75, (g).

Cases involving the latitudes and departures of a number of courses may be worked out by arrangement in successive columns, the letters designating each course written at the top (and bottom) of its respective column. The arrangement is shown in the example of a closed traverse displayed in Figure 40.

The sum of the latitudes of any closed traverse is zero. Likewise the sum of the departures of any closed traverse is zero. Or expressed in mathematical language

$$\Sigma \text{ Lat.} = 0; \quad \Sigma \text{ Dep.} = 0.$$

In traversing the field the surveyor will go as far north as he does south, and as far east as he does west. This may also be put in the familiar terms:

The total northing is equal to the total southing; and the total easting is equal to the total westing.

If the latitude (or departure) of any course is wanting, it may be found by subtracting the sum of the given north latitudes (or east departures) from the sum of south latitudes (or west departures), the algebraic difference being the missing co-ordinate.

(126) ERRORS

Should the latitudes or departures not balance, i.e., should the sum of the north latitudes be greater or less than the sum of the south latitudes, the amount of such difference is called the error in latitude; likewise any discrepancy in the departures is the error in departure.

The errors in latitude and departure having been determined, it is then necessary to find the lineal amount of error of closure. This last is the hypotenuse of the right triangle of which the sides about the right angle are the error in latitude and the error in departure. Hence the lineal error in closure is the square root of the sum of the squares of these two co-ordinate quantities. Thus lineal error of closure = $\sqrt{(\text{error in lat})^2 + (\text{error in dep})^2}$

$$E_1 = \sqrt{l^2 + d^2}$$

The error of closure is usually expressed as a ratio - the lineal error divided by the perimeter of the traverse.

$$\text{Error of closure} = \frac{E_l}{\text{perimeter}}$$

Suppose, for example, that the sum of the north latitudes and sum of the south latitudes differ by 15 links, and the sums of the east and west departures differ by 20 links, the total perimeter of the traverse being 99.12 chains; then

$$E_l = \sqrt{15^2 + 20^2} = 25; \text{ and}$$

Error of closure $= \frac{25}{9912}$, which is usually expressed in a fraction with numerator unity. This is found by dividing both numerator and denominator by the number expressing the numerator of the above fraction, E_l. In this case $25 \div 25 = 1$. $9912 \div 25 =$ 396+, or nearly 400. Hence we say, the error of closure is one in 400; i.e., $E_c = 1/400$.

This error, 1/400, is a very large error. In compass surveys, the error of closure should never exceed 1/500, and is seldom greater than 1/2000. A transit survey error of closure should evidently be much less than this; 1/10000 is a maximum, and often, as in city surveys, 1/40,000 is sought.

The error of closure may be due to errors in angle measurement, which usually occur when the angles measured lie between lines that are short, or when measuring angles with the compass. (See Assignment VI, Compass Surveying.) The error may result from difficulty in measuring lines, a failure to keep the tape

taut, or failure to level it over uneven ground. Also, mistakes may occur in reading the tape, although these may usually be detected from gross discrepancies that should be removed by remeasuring the courses in which such mistakes seem to occur.

(127) ADJUSTING LATITUDES AND DEPARTURES

Before proceeding to use latitudes and departures in computations, or for mapping, they should be adjusted proportionally, so that the northings and southings, and eastings and westings balance. This is generally called "balancing the survey", and the adjusted co-ordinates are called "balanced latitudes", "balanced departures".

Balancing is accomplished by distributing the error in latitude and also the error in departure proportionally to their respective co-ordinates. The errors in line measurement usually result in measuring the line <u>too long</u>, as may be seen when we consider that errors here arise from sag, alignment, and lack of tension. Therefore it is better practice to deduct from the greater side than to add to the lesser.

<u>Errors in Angle Measurement</u>. Where measurements of angles are made with compass the error may be large, 5' to 15', throughout the course. Such error should be distributed in parts to the respective angles in which error is most likely to occur. Angles measured by the transit are more precisely determined, and differ by a few seconds or one or two minutes; these errors when small may be applied to the most doubtful angle. In any case where the

error in angle shows a gross mistake (a blunder in reading), as for example where angle measured does not check with magnetic bearings, a remeasurement of angle should be made.

In all these considerations it is necessary that every angle and every line should be measured in the field, as any such part omitted while it may be supplied by computation explained further on (Assignment XVIII), no check can be had upon the work and hence the attempt to adjust latitudes and departures is futile.

In balancing the latitudes and departures of a compass survey the following rule is applied:

Having determined the latitude error, subtract such proportional part of the error from each computed latitude as the length of its course bears to the perimeter of the field. Also to adjust departures subtract a proportional part of the error indeparture from the departure of each course. The corrections in all cases should be applied in such a way as to reduce the difference between the sums of the two columns, so that the resulting difference is zero; i.e., the sums balance.

In a transit survey the correction is made by proportional parts of the error in the latitude or departure to each course, as compared with the total latitude or total departure without regard to algebraic signs.

This subject may best be explained by taking a concrete example and following the computations, step by step. As such example, we take the data of the following traverse, a chain and

compass survey of a quadrangular field. The data given are the course, bearing and distance.

Course	Bearing	Dist.	N.Lat.	S.Lat.	E.Dep.	W.Dep.	Balanced Lat.	Balanced Dep.
A - B	N45°30'E	2.87	2.01		2.05		+2.01	+2.05
B - C	S51°10'E	5.05		3.17	3.93		-3.19	+2.93
C - D	S58°10'W	4.63		2.44		3.93	-2.45	-3.94
D - A	N29°15'W	4.16	3.63			2.03	+3.63	-2.04
	Totals	16.71	5.64	5.61	5.98	5.96	0.00	0.00
			5.61		5.96			

Error in Lat. 0.03 0.02 = Error in Dep.

Error of closure = $\sqrt{.03^2 + .02^2}$ = 0.036; i.e., 1/464

The logarithmic computations are given for the latitudes and departures recorded in the above table in the following scheme:

Course	A - B	B - C	C - D	D - A
Lat.	2.01	3.17	2.44	3.63
log Lat.	0.30354	0.50060	0.38776	0.55985
log cos	9.84566	9.79731	9.72218	9.94076
log Dist.	0.45788	0.70329	0.66558	0.61909
log sin	9.85324	9.89152	9.92921	9.68897
log Dep.	0.31112	0.59481	0.59479	0.30806
Dep.	2.05	3.93	3.93	2.03
Course	A - B	B - C	C - D	D - A

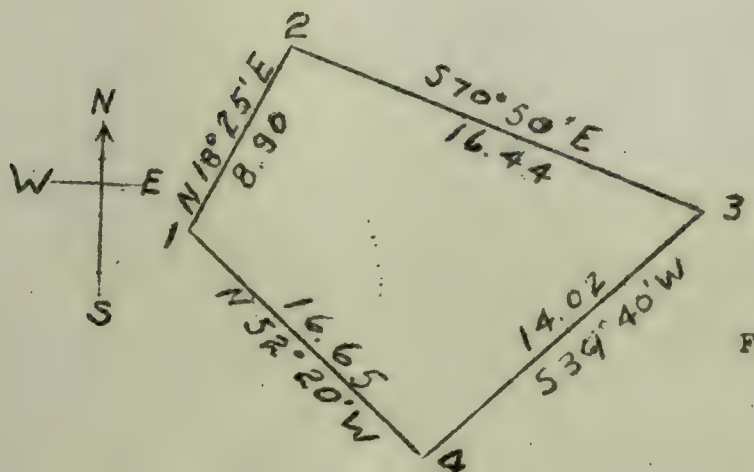


Figure 41

Problem:

As an exercise in computing and tabulating the data, in the computation of latitudes and departures, and in balancing them use the data given in Figure 41.

REFERENCES:

Tracy	Pages	384 - 398
Raymond	"	141 - 148
Johnson	"	185 - 193
Breed & Hosmer	"	400 - 408, Vol. I

UNIVERSITY OF CALIFORNIA EXTENSION DIVISION
CORRESPONDENCE COURSES IN ENGINEERING SUBJECTS

Course 1A Elements of Surveying Swafford

Assignment 17

Double Meridian Distances - Areas

FOREWORD

This assignment will explain the meaning and computation of Double Meridian Distance, Double Parallel Distances, and the Computation of Areas.

(129) DEFINITIONS

The Meridian distance of a line is the horizontal distance of its middle point from the meridian (line) of reference. This meridian of reference may be the magnetic meridian, the true meridian, or any assumed line.

In Figure 42, the lime MN has a meridian distance equal to Ch which joins the mid-point of MN and is perpendicular to the meridian of reference XY. Ma is the north latitude, aN the east departure. But Ch = 1/2 aN; i.e., the departure of the course MN is double the meridian distance (D.M.D.) as defined above. Hence to find the double meridian distance of the first course take the departure of that course. D.M.D. of MN = aN. The meridian distance of the course NO is bj, the perpendicular from the mid-point

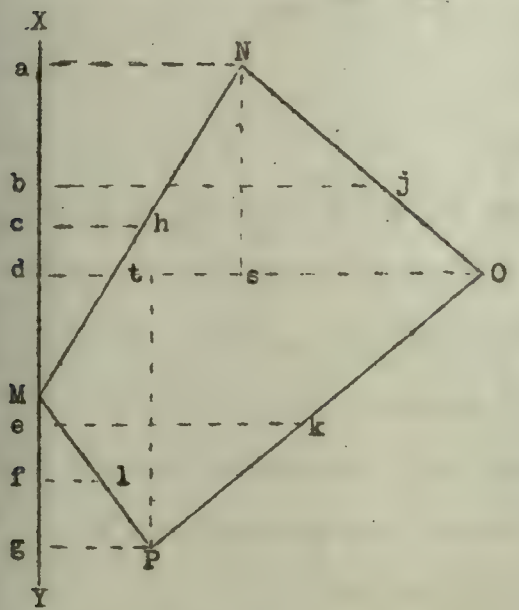
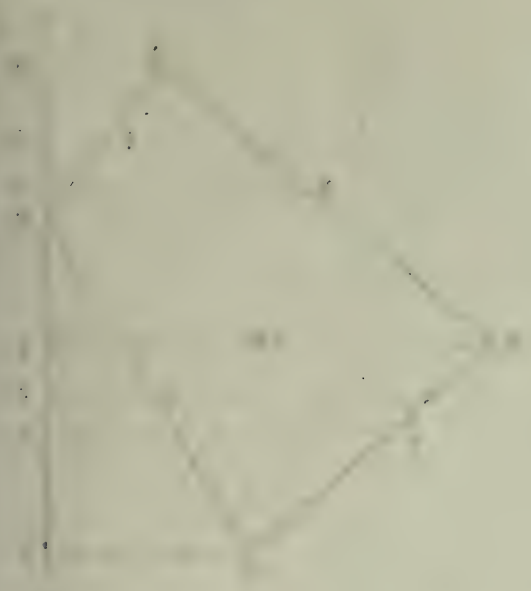


Figure 42.



of NO to the meridian of reference XY. But bj = 1/2 (aN + dC), since the line joining the mid-points of the non-parallel sides of a trapezoid is parallel to the bases and equal to one-half the sum of the bases. That is, aN + ds + sO = 2 bj (since dO = ds + sO). In other words, the D.M.D. of the second course is equal to the departure of the first course plus the D.M.D. of the first course plus the departure of the second course.

D.M.D. of NO = aN + aN (= ds) + sO.

The meridian distance of OP is ek, the perpendicular from the mid-point of OP to XY. ek = 1/2 (gP + dO), which is to say, that ek joining the mid-points of the non-parallel sides of the trapezoid dOPg is equal to one-half of the parallel sides. Hence, the D.M.D. of OP = 2ek = gP + dO. But gP = dt = dO - tO, and dO = aN + sO. Therefore, by substitution,

D.M.D. of OP = dO - tO + aN + sO

= aN + sO - tO + aN + sO

= aN + aN + sO + sO - tO

But aN + aN + sO = D.M.D. of the second course, and -tO = departure of the third course, which is a west departure, and therefore negative. Lastly, by construction, fl is the meridian distance of the course MP and is equal to 1/2 gP. Consequently the D.M.D. of the last course is , like the D.M.D. of the first course, equal to the departure of that course. Or thus:

D.M.D. of PM = 2 fl = gP = dt.

Computing from the previous course,

D. M. D. of $\underline{PM}$ = D. M. D. 3rd C + Dep. 3rd C + Dep. 4th C.

$= 2\underline{aN} + 2\underline{sO} - \underline{tO} + (-\underline{tO}) + (-\underline{gP})$, but $\underline{gP} = \underline{dt}$

$= 2\underline{dO} - 2\underline{tO} - \underline{dt}$

$= \underline{dt} = \underline{gP}$

This value, $\underline{gP}$, you will note, is equal to the departure of the last course but of contrary sign. This is true of any closed traverse and is a convenient check upon the computations of the D. M. D.'s and should always be observed to that end. Of course it is required that the latitudes and departures are adjusted; i.e., balanced.

We will now take up the specific case of a simple traverse, showing the computation of the double meridian distances.

Course	Bearing	Dist.	N. Lat.	S. Lat.	E. Dep.	W. Dep.	D. M. D.
A - B	S35°32'E	8.6 ch.		7.0	5.0		5.0
B - C	N71°34'E	9.5 "	3.0		9.0		19.0
C - D	N18°26'E	12.7 "	12.0		4.0		32.0
D - E	Due West	11.0 "	0.0			11.0	25.0
E - A	S41°11'W	10.6 "		3.0		7.0	7.0
			15.0	15.0	18.0	18.0	

Balanced

It will be noted that the latitudes and departures balance, and we shall now proceed with the computations as follows:

5.0	D.M.D. of 1st course (= Dep. of that course)
5.0	Dep. of 1st course
9.0	" of 2nd "
19.0	D.M.D. of 2nd course
9.0	Dep. of 2nd course
4.0	" of 3rd course
32.0	D.M.D. of 3rd course
4.0	Dep. 3rd course
-11.0	Dep. 4th course
25.0	D.M.D. of 4th course
-11.0	Dep. 4th course
- 7.0	Dep. 5th course
7.0	D.M.D. 5th course. This is numerically equal to the

departure of the 5th (last) course, but of contrary sign. Therefore, the computation checks.

(130) RULE FOR COMPUTING D.M.D.'s

(a) The D.M.D. of the first course is the departure of that course.

(b) The D.M.D. of the second course is the sum of the departure of the first course, the D.M.D. of the first course, and the departure of the second course.

(c) The D.M.D. of the nth course is the sum of the D.M.D. of the (n-1)th course, the departure of the (n-1)th course, and the departure of the nth course.

(d) The D.M.D. of the last course is numerically equal to the departure of the last course, but of contrary sign.

(131) AREAS

By referring to the traverse MNOPM, Figure 42 of this assignment, it will be seen that the figure there shown is composed of the

trapezoids aNOd and dOPg, which together include the area of the field plus the areas of the triangles aNM and MPg.

The area of the trapezoid is equal to the product of the meridian distance into the latitude, or twice the area of each trapezoid is the product of the D.M.D. of any course into the latitude of that course. It will be seen that certain of these products are positive and others negative.

By choosing the meridian of reference, XY, through the extreme west point, M, the resulting signs of all the D.M.D.'s are positive. Hence, the products of D.M.D.'s and south latitudes will be minus areas, and the products of D.M.D.'s and north latitudes will be plus areas. The difference between the minus areas and plus areas will therefore be the area of the field. This then gives us the Rule for Computing the Area of the Field from Latitudes and D.M.D.'s as follows:

(a) Multiply the latitude of each course by its D.M.D. having regard to sign.

(b) The algebraic sum of the products is the double area; divide by 2 to find the area.

If the course distances have been measured in chains, the products will be square chains. This may be reduced to acres by dividing by 10, since 10 sq. ch. = 1A. Simply remove the decimal point one place toward the left.

Had the lines of the traverse been measured in feet, the resulting area would have been square feet; and since 43,560 sq. ft.

equal 1 A, to reduce to acres, divide by this number (43,560).

The following is a tabulation of the latitudes, departures, D.M.D.'s and products of the five course traverse contained on a previous page:

Course		Lat.	Dep.	D.M.D.	+ Prod.	- Prod.
A - B	Bearings and Distances omitted	- 7.0	+ 5.0	5.0		35.00
B - C		+ 3.0	+ 9.0	19.0	57.00	
C - D		+ 12.0	+ 4.0	32.0	384.00	
D - E		0.0	- 11.0	25.0	0.00	
E - A		- 8.0	- 7.0	7.0		56.00
				Totals	441.00	91.00
					91.00	
					2)350.00	
					175.00 sq. ch.	
					17.5 acres	

We now give the following data, showing a complete tabulation of bearings, distances, etc. of the traverse, of a quadrangular field:

ourse	Bearing	Dist.	N. Lat.	S. Lat.	E. Dep.	W. Dep.	Lat.	Dep.	D. M. D.	+D. Area	-D. Are
A - B	N84°00'W	9.04	0.945			8.990	+ 0.95	- 8.98	- 8.98		8.531
B - C	S21°15'W	12.34		11.501		4.472	-11.50	- 4.45	-22.41	257.7150	
C - D	N72°15'E	12.92	3.939		12.302		+ 3.95	+12.32	-14.54		57.433
D - A	N 9°30'E	6.68	6.588		1.105		+ 6.60	+ 1.11	- 1.11		7.326
		40.98	11.472	11.501	13.407	13.462	balanced			257.7150	73.290
				11.472		13.407				73.2900	
				0.029		0.055				2)184.4250	
				Error in Lat.		Error in Dep.				92.2650 = 9.22	

Error in Closure = $\sqrt{(.03)^2 + (.06)^2}$ = .067, or 1/600, about.

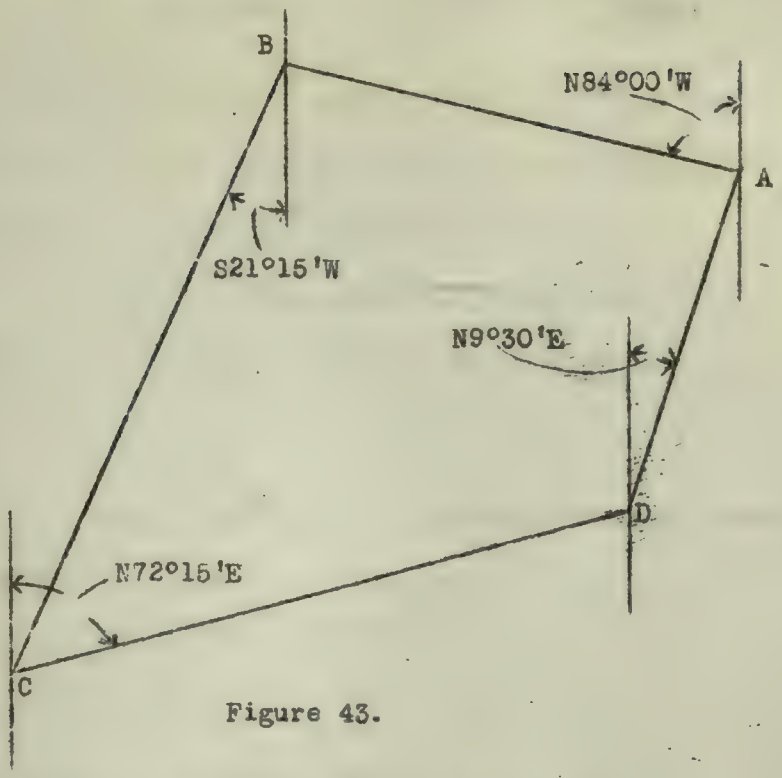


Figure 43.

Interior Angles

A = 86°30'
B = 105°15'
C = 51°00'
D = 117°15'
360°00'

Logarithmic Computation of Latitudes and Departures

Courses	A - B	B - C	C - D	D - A
Lat.	0.945	11.501	3.939	6.588
log Lat.	1.97540	1.06074	0.59537	0.81878
" cos	9.01923	9.96942	9.48411	9.99400
" Dist.	0.95617	1.09132	1.11126	0.82478
" sin	9.99761	9.55923	9.97882	9.21761
" Dep.	0.95378	0.65055	1.09008	0.04239
Dep.	8.990	4.472+	12.302	1.105
Courses	A - B	B - C	C - D	D - A

Computation of D.M.D.'s

- 8.98	D.M.D.	1st C.
- 8.98	Dep.	" "
- 4.45	Dep.	2nd "
-22.41	D.M.D.	2nd "
- 4.45	Dep.	" "
+12.32	Dep.	3rd "
-14.54	D.M.D.	" "
+12.32	Dep.	" "
+ 1.11	Dep.	4th "
- 1.11	D.M.D.	4th C.

The D.M.D. of this last course is numerically equal to the Dep. of the last course, but of contrary sign. CHECK

Here we have placed the meridian of reference through A, at the extreme east point and using the usual convention, east positive,

west negative, the resulting D.M.D.'s are all negative. Had the meridian been passed through C, the D.M.D.'s would then be positive; or, if through B or D, some would then be positive, some negative. The usual effect of such assumptions as the last two, is to complicate computations and this should be avoided.

PROBLEM 1

As an exercise for the student, the following data are given:

Station	Azimuth	Distance
A	107°15'	16.4 ch.
B	196°15'	24.1 "
C	246°05'	19.6 "
D	13°55'	37.0 "

Reduce azimuths to bearings, compute latitudes and departures, and balance them; also compute D.M.D.'s and areas in acres.

Show errors in latitude and departure and lineal error of closure; also the error ratio.

Sketch the field approximately to scale and find the interior angles. Use every means of checking your results.

In conclusion you are referred to other methods of computing areas already given in Assignment XV.

(132) DOUBLE PARALLEL DISTANCES

Instead of D.M.D's. we may use double distances from the parallel (or horizontal co-ordinate axis) and for computation of areas multiply these by the departures in a manner similar to the preceding method. In fact, where it is desirable to check computations, this affords a valuable means, and it is sometimes so employed.

In cases where D.P.D's. (double parallel distances) are used, it is convenient to pass the horizontal co-ordinate (which may be called a parallel of reference) through the most southerly point and thus all D.P.D's. will be positive as they will be measured northward which, by convention, is regarded as plus.

PROBLEM 2 As an illustrative example of this method and for clearly fixing the manner of computation of D.P.D.'s and also comparing results with those obtained from D.M.D's., the student is here given a simple traverse which should be worked out both ways.

Course	Bearing	Distance	Balanced Lat.	Dep.	D.P.D.
A - B	S 69°15' E	7.06 ch.			
B - C	N 37 15 E	5.93			
C - D	N 39 30 W	6.00			
D - E	S 57 45 W	4.65			
E - A	S 30 00 W	4.98			

REFERENCES:

Tracy	Pages	415-416
Raymond	"	152-153
Johnson	"	185-193
Breed & Hosmer	"	400-404, Vol. I

Course 1A Elements of Surveying

Assignment 18

SUPPLYING OMISSIONS - PARTING OFF LAND

FOREWORD

Certain data wanting in any land traverse may be supplied by computations from the given data under allowable conditions, and a portion of this assignment will treat of a few cases of such nature. The subject of Land Surveying as treated in this course will give also a few typical cases of dividing or parting-off land. Problems of both classes may be given in many forms, and therefore only a limited number of each will be taken up.

(132) MISSING DATA

As stated in the treatment of latitudes and departures, the sum of the north latitudes is (in any closed traverse) equal to the sum of the south latitudes; also the total east departure is equal to the total west departure.

Regarding the north latitudes as positive and the south latitudes as negative, the east departures as positive and the west departures as negative, it follows that the algebraic sum of latitudes is zero, and the algebraic sum of the departures is zero.

Hence $\sum$ Lat. = 0; $\sum$ Dep. = 0. Or to write in extended form:

$$A \cos \alpha + B \cos \beta + C \cos \gamma + D \cos \delta = 0 \quad (a)$$

$$A \sin \alpha + B \sin \beta + C \sin \gamma + D \sin \delta = 0 \quad (b)$$

where A, B, C, and D are the distances of the several courses and α, β, γ, and δ are the respective bearings.

We thus have two simultaneous equations and are therefore able to solve for two unknown quantities, whether they be:

1. Length and bearing of one course.
2. Length of one and bearing of another course.
3. Lengths of any two courses.
4. Bearings of any two courses.

(133) Suppose the distance, D, and the bearing of the same course, δ, are unknown. (Figure 44)

Then D cos δ will equal the algebraic sum of the latitudes of the remaining courses with their signs changed; in other words, it is the value of the unknown term in equation (a) above.

In like manner D sin δ is also found from equation (b).

The course distance is now computed by the formula $Dist^2 = Lat^2 + Dep^2$; also the bearing D-A is the angle whose tangent is $\frac{Dep}{Lat}$ (i.e. $\frac{D \sin \delta}{D \cos \delta}$). Or having computed the distance, D, the sine and cosine may both be found; the one by dividing the departure by D; the other by dividing the latitude by D. This last method furnishes a convenient check upon computations.

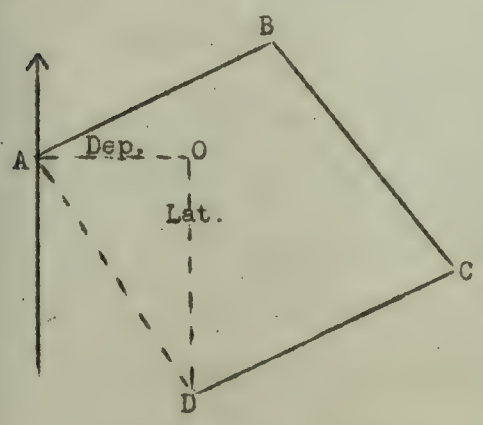


Fig. 44

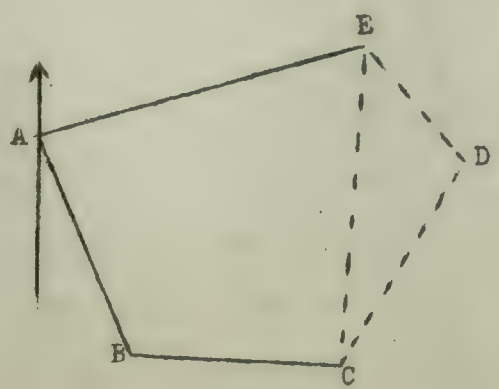


Fig. 45.

(134) If the missing parts are two adjacent sides, as CD and DE in Figure 45, compute the length of an auxiliary line, CE, by the above method; then in the triangle CDE the parts CD and DE may be readily found, since the side CE and the angles E and C are known.

(135) When the missing parts are two non-adjacent course distances, (Figure 46), the following offers an ingenious solution: Given the traverse ABC--A, in which all sides and bearings except the non-adjacent sides AG and BC are known; to find AG and BC.

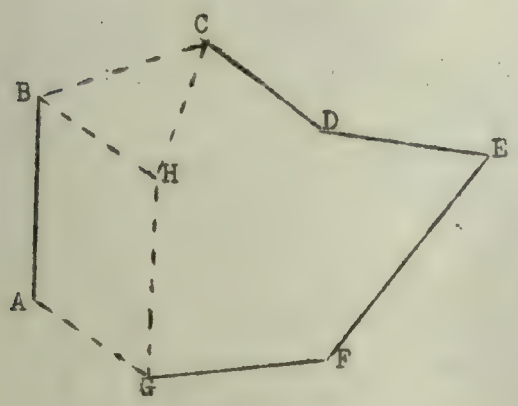


Fig. 46

Solution: Construct HG parallel and equal to AB and draw HC, thus forming the closed traverse CDEFGHC. By Case I compute HC.

In the triangle BHC we now know its angles and the side HC, from which BC and BH may be readily computed. BH is equal to AG (the opposite sides of a parallelogram).

Each problem of this nature requires a little study to determine the best method of attack, but if the unknown parts do not exceed two, the solution is possible. Indeed there are certain special classes when even three unknowns may be found by readjustment of figure, but it is not required to enter upon an explanation of such cases.

These methods are useful in the parting-off of land treated in the following pages and a reference will be made to this part of the assignment in the explanations.

(136) PARTING OFF AND SUBDIVIDING LAND

Most of the problems in the parting off and subdividing of land are purely geometric. Many others may be solved by the simple methods of trigonometry, but a few must be dealt with by resort to co-ordinate means similar to the foregoing.*

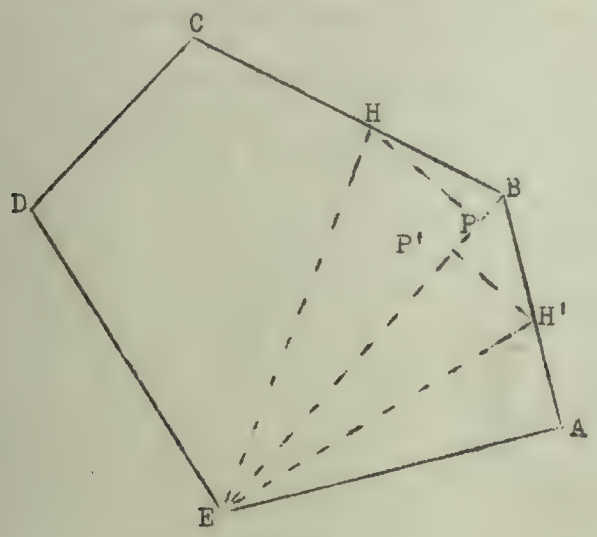


Figure 47

Let it be required to cut off a given area from the field ABCDE by a line starting at the corner E.

First cut off the triangle EBA and compute the distance EB and its bearing. Deduct the area of triangle EBA from the required area; the residue will be the area of triangle EBH, and it is therefore required to find H in BC. Hence find HP, the perpendicular upon EB. Area of $\triangle$ EBH $= \frac{BE \times HP}{2}$

$$\therefore HP = \frac{2 \times \text{area} \triangle EBH}{BE}$$

We now have the angle HBP, angle BPH, and angle PHB, also the perpendicular HP; from these the distance BH may be computed. Also compute the angle BEH, i.e., the bearing of EH. With transit

* You will find many examples of this nature in text books on surveying: the work of Gillespie and that of Carhart are specially cited.

or compass run the line EH and check the falling of the line on BC by measuring the distance HB.

Note: If the area of EAB is greater than the required area to be cut off, construct the triangle EH'B and compute BH'.

It is sometimes required to run a line perpendicular to a given side of a field as a road skirting the side of such field and to part off by this means a definite area. Suppose that a certain number of acres are to be cut off from the right hand portion of the field MNOPR by a line perpendicular to the road extending along MR (Figure 48).

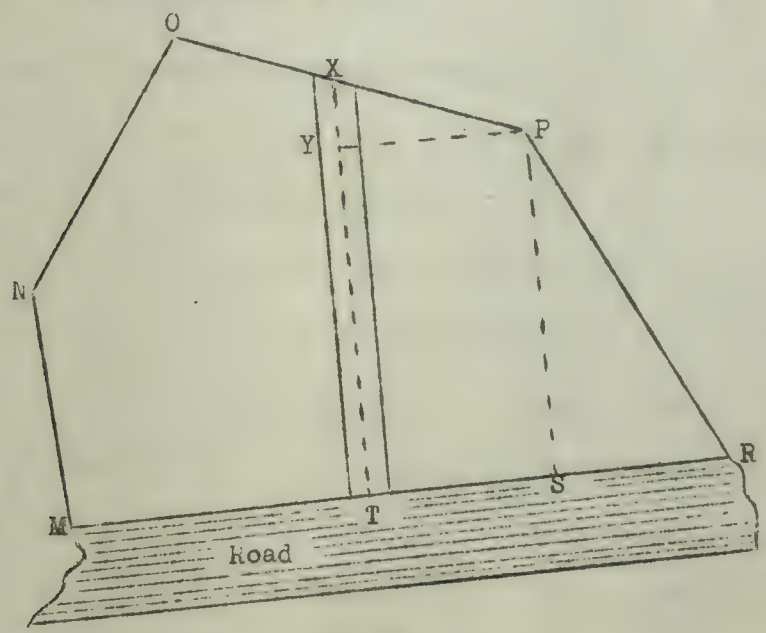


Figure 48

Erect the perpendicular PS forming the triangle RSP and having computed the area deduct it from the required area. Assume a point T and draw TX also perpendicular to MR. The area of the trapezoid TSPX may now be measured and computed. If this area

differs from the residual area as found above, adjust the line TX by successive trials until the small difference becomes less than any appreciable quantity. This may be done by office computation and the true line run in the field.

Had it been required to run the dividing line at a given angle other than perpendicular, a trial line might have been run from a point, say, on MR having the required bearing, the area of the part so cut off computed and deducted from the required area. Then trial lines run at such distance from the first cut-off line, adjusting the same until the area equals the residual area or differs by an inappreciable quantity.

A field having an irregular bounding line as the meander in Figure 49 is to be divided by a line, as KM, running from the point K and cutting off a given area.

In this case the survey of the field should be made to include the line BC and a series of off-sets taken to the meander line. Compute the area of the portion bounded by BC and the meander line and deduct this value from the required area to be cut off.

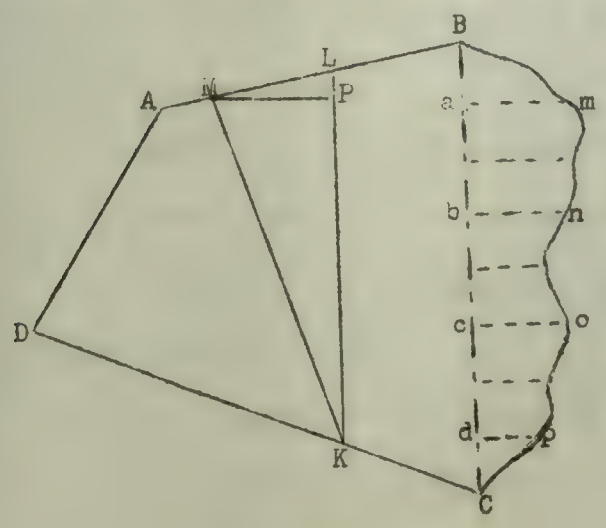


Figure 49

From K run KL parallel to BC, forming the trapezoid KLBC and

compute the area of the trapezoid deducting this much also from the required area. The residual area is now that of the triangle KLM; KL being computed from the foregoing, MP and ML may be found by using the area of the triangle KLM and the bearing of AB.

If the part bounded by the straight line BC and the meander BmnopC be considered, it is readily seen that the area of this section is approximately the sum of the areas of the several trapezoids composing it. Taking the distances between off-sets equal along BC, then BC (an+bm+co+dp) will equal the area of the irregular section.

(137) More accurate formulae for computing the areas of irregular boundary sections are

Area = 1/2 l $(h_1 + h_2)$ for each trapezoid, l being a segment along BC, and h_1, h_2 being the off-sets at two successive points. This formula is known as the Trapezoidal Rule.

Simpson's 1/3 Rule.

In this three off-sets at regular intervals are taken and the formula becomes:

Area = 1/3 l $(h_1 + 4h_2 + h_3)$.

Substituting L for 2 l in the above formula we have

L/6 $(h_1 + 4h_2 + h_3)$ which is the well known Prismoidal Formula applied to areas.

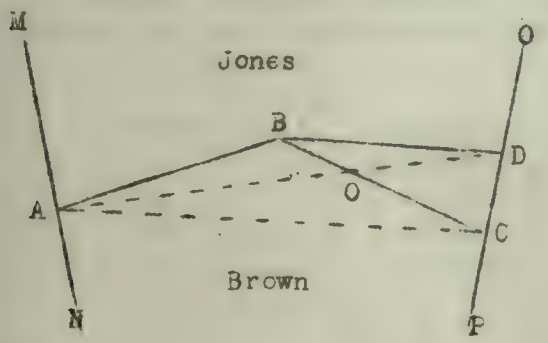
Still another formula known as Simpson's 3/8 Rule is:

Area = 3/8 l $(h_1 + 3h_2 + 3h_3 + h_4)$.

(138) CHANGING BOUNDARY LINES

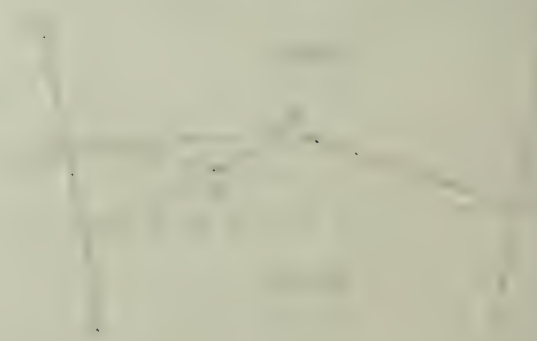
It is sometimes desirable to change the boundary or divisional lines of property, as in straightening a crooked line between adjoining properties, or placing a line to alter the boundary without changing the content. To illustrate this we give a common case of the adjusting of the divisional line between properties, the area remaining the same.

The line between the property of Jones and Brown as shown in Figure 50 is to be made straight by a line starting at A and run in such a way as to make no alteration in the land area of the property of either owner.



The bearing and the distances of the adjacent lines, as MN, OP and also of AB, BC, are known and it is first required to find the bearing of the auxiliary line AC. Then from B run a line parallel to AC (i.e., having the same bearing). Compute the back bearing of AD and run a line from D to A.

The triangles ABC and ADC are equal in area having a common base AC of equal altitudes - the vertices B and D falling on a parallel to the base. Hence the line AD is the line required. It is also evident that the triangle AOB is equal in area to COD, and that by the establishment of the new divisional line Jones receives from Brown as much as he yields by this adjustment.



QUESTIONS:

1. Given the following transit traverse; calculate the error of closure and balance the traverse. Assume a reference meridian through point A and calculate the D.M.D. of each course from this meridian. Find the area in acres.

Course	Bearing	Distance (feet)
AB	S 19° 33' E	267.50
BC	S 44 12 E	301.89
CD	N 22 42 E	266.80
DE	N 60 02 E	323.61
EF	N 8 55 W	195.65
FG	S 69 19 W	354.50
GA	S 88 36 W	321.20

Note: In solving this problem use the standard forms prepared by the Department of Civil Engineering. Submit original computations, not a copy.

2. In the following notes of a compass survey, the length and bearing of one of the courses were omitted. Supply the missing data.

Course	Bearing	Distance (chains)
AB	N 30° 30' E	28.37
BC	S 64 20 E	34.21
CD	S 13 15 W	41.90
DA	omitted	omitted

REFERENCES:

Tracy	Pages 421 - 427
Raymond	" 163 - 165
Johnson	" 213 - 219
Breed & Hosmer	" 414 - 419, Vol. I

UNIVERSITY OF CALIFORNIA EXTENSION DIVISION
CORRESPONDENCE COURSES IN ENGINEERING SUBJECTS

Course 1A Elements of Surveying

Assignment 19

SURVEY OF THE PUBLIC LANDS.

(139) FOREWORD

In 1785 the Congress of the United States by enactment decreed that the public domain should be divided by survey into townships six miles square, containing 36 sections, each one mile square. This seemingly simple plan carried with it so many unlooked-for complications, that an elaborate scheme was required to carry out the ideal conception of covering a spherical surface with rectangular tracts. We need not follow the evolution of this scheme. The purpose of the Assignments 19 and 20 is to show how the present methods are applied in Public Land Survey.

In general, then, the scheme provides for the laying out of meridiams and parallels of latitude primarily determined by an initial point carefully chosen and established. The rectangular subdivisions are laid out to conform with these primary lines.

(140) INITIAL POINTS

Initial points from which the lines of the Public Surveys are to be extended are established whenever necessary under direction of the Commissioner of the General Land Office. These points are selected with a view to the survey of extensive agricultural areas within reasonable geographical limitations. The position of an initial point in latitude and longitude is determined by accurate field astronomical methods.

Three such points in California are Mt. Diablo, long. 121°54'48", lat. 37°51'30"; Humboldt, long. 124°07'11", lat. 49°25'04"; San Bernardine, long. 116°56'15", lat. 34°07'10". Suitable monuments marking these points have been erected on Mt. Diablo, Mt. Pierce (Humboldt), and Mt. San Bernardino as affording at once a permanent and prominent mark from which the primary lines are extended.*

(141) PRINCIPAL MERIDIAN

This line is extended north and south through the initial point and conforms with the true meridian. On the principal meridian are set quarter section corners and section corners at 40 and 80 chains respectively, and regular township corners at 480 chains. Meander corners are set at all meanderable bodies of water, as streams, lakes, etc. Such meridional lines are rigorously determined by astronomical means. The line measurements are made by refined methods, usually two or more times for check. Every reasonable effort is exercised to insure the accuracy of the alignment and the measured lengths. If the tests show a deviation of 3'00" or a discrepancy of more than 20 links in 80 chains (1 mile), then the line is remeasured to reduce the error and also the line is changed in azimuth to reduce the error in alignment. The establishment of a Prinoipal Meridian being fundamental, the precise determination is essential as it affects all surveys related to it.

(142) BASE LINE

From the initial point the Base Line is extended east and

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* For a complete list of Initial Points see Manual of Surveying Instructions, Chap. III, issued by the General Land Office, Washington, D. C.

west on a true parallel of latitude. Upon this line, as upon the Principal Meridian, quarter-section and section corners are established at intervals of 40 chains and 80 chains respectively and standard township corners at 480 chains; also meander corners where the line intersects bodies of water.

A straight line as projected through any point in an east and west direction is the arc of a great circle of the sphere (the earth). It will not be coincident with a parallel of latitude, but may be conceived as a tangent at the common point from which the parallel may be run through points determined by off-sets. This is known as the Tangent Method of running a parallel. It is conveniently used in open country sparsely covered with forest or underbrush, where thick brush and close-set trees do not interfere seriously with the measuring of off-sets. If the country is not open, another and more expeditious method, known as the secant method, of running parallels of latitude is employed. The secant, also the arc of a great circle, is run from a point south of the initial point or township corner, which is carefully determined, the distance from the corner being a function of the latitude. The secant is then projected east or west as the case may be. It will cut the parallel at the one mile and five mile points and off-sets will locate the other mile points on the parallel. This and the tangent method will be fully explained in Assignment XX.

If meridional lines are accurately run at the mile and quarter-section (1/2 mile) points and east and west lines run through

such meridians, a series of such east and west lines will lie approximately in a curve which follows the parallel of latitude for each point. To lay out a parallel by such method requires, however, that meridians be established by strict astronomical means and that a small correction to alignment be applied at each mile point on the east or west line. In some cases, where from the nature of the territory surveyed, neither the tangent nor the secant method is conveniently applicable, then the "solar method", as this third method is called, may be preferable.

(143) STANDARD PARALLELS

These are parallels of latitude which run in the same manner as the base-line at intervals 24 miles apart, north and south of the base line. The 24 mile intervals are chosen as they include 4 intervals of 6 miles each which is a dimension of the township. Care is exercised in laying out standard parallels as they constitute lines of control in the subdivision of the 24 mile tracts into townships.

(144) GUIDE MERIDIANS

Guide meridians are extended north and south from the base line at intervals of 24 miles east and west of the principal meridian. As all meridians are not parallel lines, but converge toward the pole, they will not intersect the standard parallel north or south at intervals of 24 miles, but the intersection of the guide meridian with the standard parallel is marked and such a point is named a "Closing township corner". Also in running guide

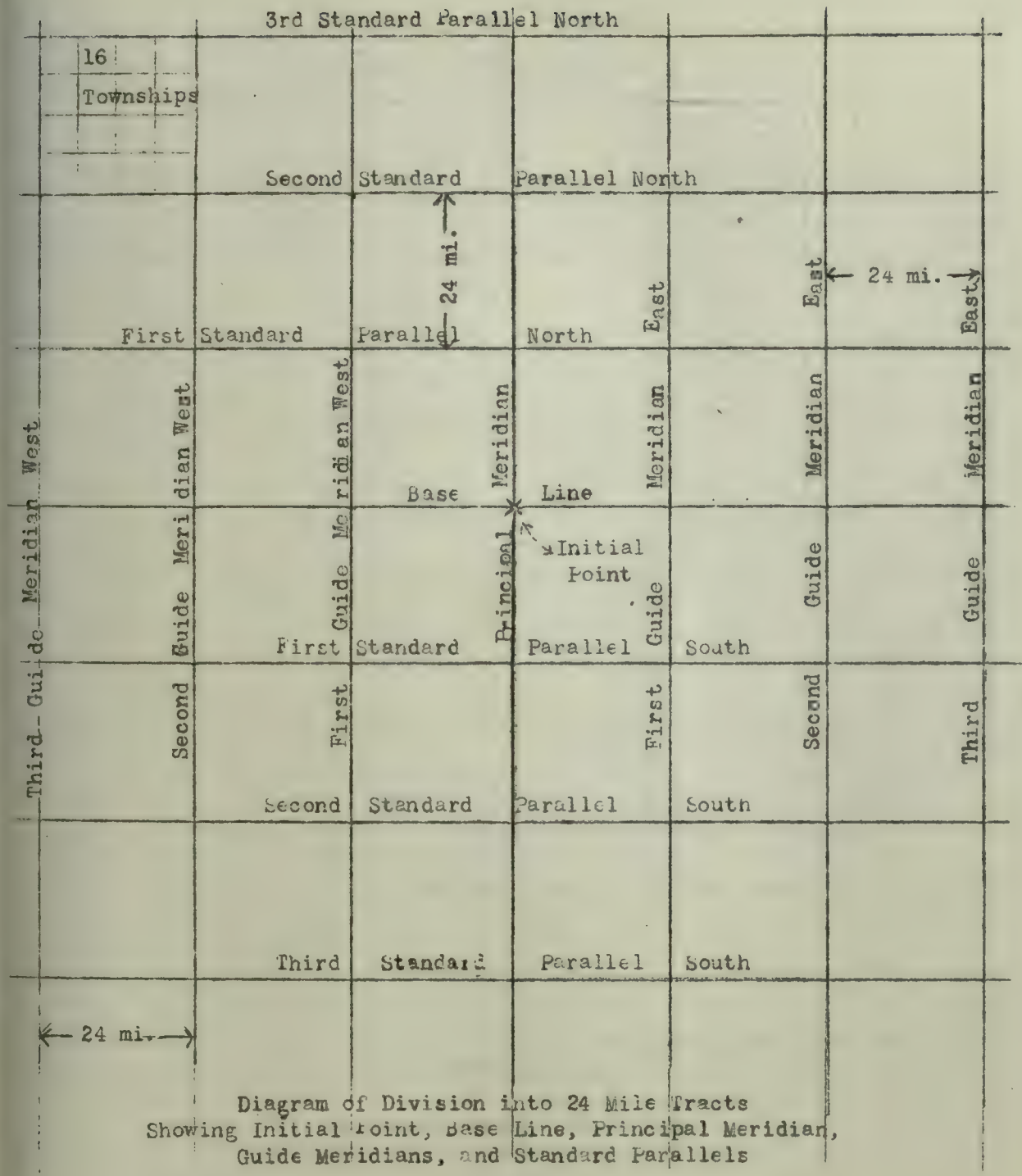

Diagram of Division into 24 Mile Tracts
Showing Initial Point, Base Line, Principal Meridian,
Guide Meridians, and Standard Parallels

Figure 51.

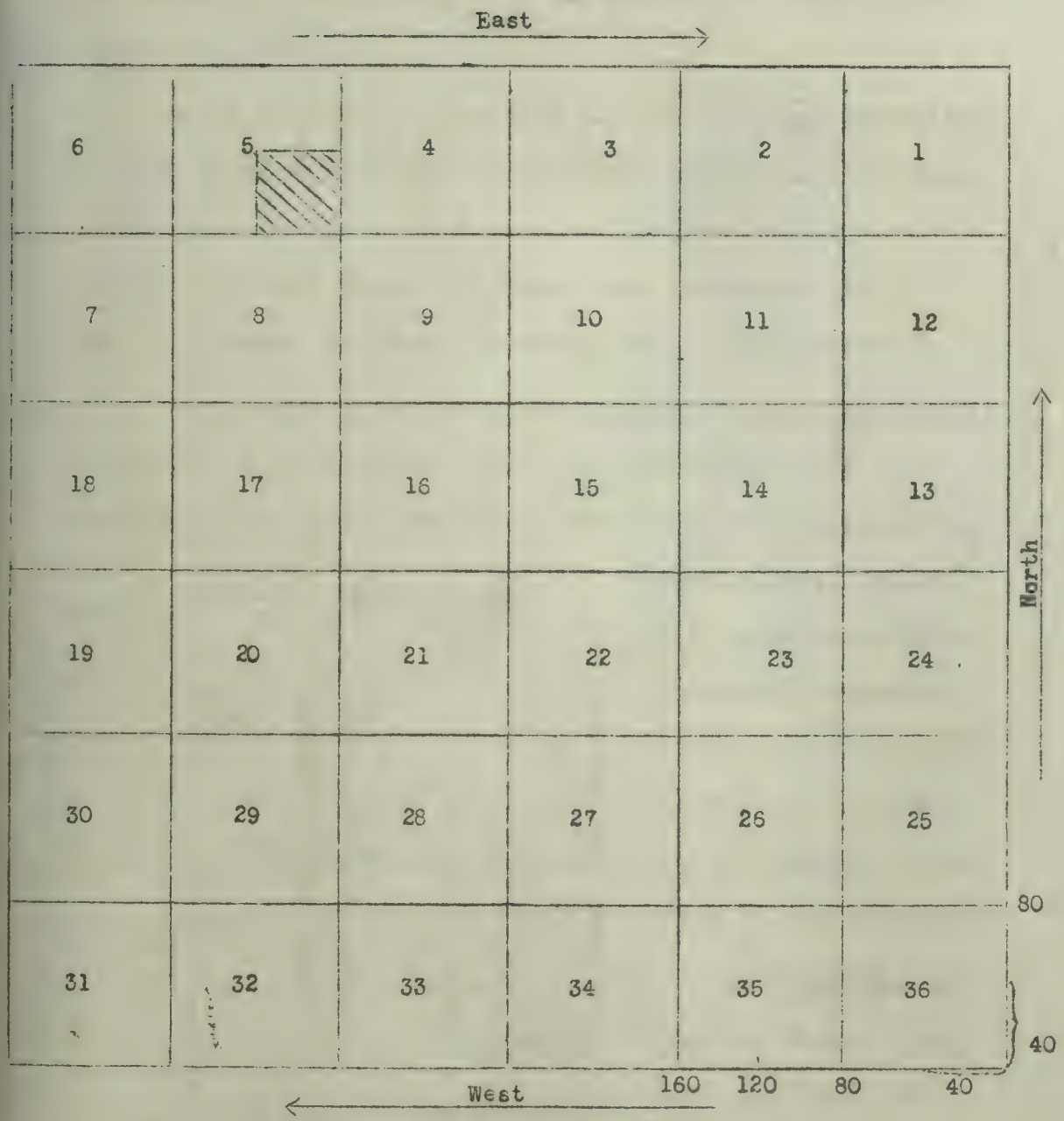

Diagram of Township subdivided showing order of numbering Sections.

Figure 52

meridians south of any base line, the linear convergence is set off on the base line from computed values, thus causing the standard parallel 24 miles south to be subdivided into township distances of 6 miles.

Where intervals of more than 24 miles have been established, as in certain old surveys, a new standard parallel or other guide meridians may be run to which a special designation is given: such as "auxiliary parallel"; "Cedar Creek Correction Line"; "Auxiliary Guide Meridian"; or "Grass Valley Guide Meridian".

(145) The diagram of 24 mile tracts, Figure 53, shows the Initial Point, Base Line, Principal Meridian, Standard Parallels, and Guide Meridians. These constitute the primary subdivision of the Public Lands. The 24 mile tracts are subdivided into townships 6 miles square and these in turn into sections one mile square, 36 sections in each township.

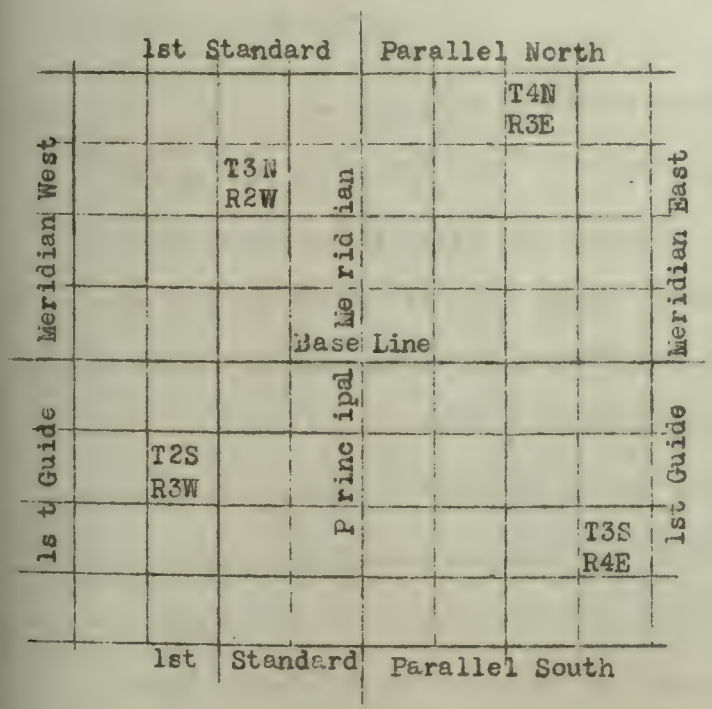

Figure 53

The townships are numbered in each range north and south, the tiers of townships constituting ranges. The

ranges are designated as range 1 east (R1E) or range 3 west (R3W) etc. The township numbering is Twp.2N,R4E, etc.

The survey of townships begins at a corner on a meridional line. The east and west boundaries are surveyed as true meridians, and section and quarter-section corners are established at each 80 and 40 chain interval. The township corners are also established at 480 chains and meander corners at meanderable bodies of water. The parallel boundaries falling upon a base line or standard parallel or a meridional line upon an established guide meridian or principal meridian need not be relocated, but such lines and locations must be checked to verify measurement and location by a former surveyor, and if found to be correct, they are entered in the notes of the subdivisional survey.

(146) Let it be required to subdivide the 24 mile tract immediately to the north and west of a base line and principal meridian. The surveyor checks first the principal meridian, by running one mile or more north from the initial point, and then the standard parallel (in this case the base line), by retracing the line to the west from the initial point and compares the bearings and distances observed with those of the original survey of the 24 mile tract. He then continues to the southwest corner of township 1 range 1 west (which is the southeast corner of township 1 range 2 west) and having set up his instrument at this corner proceeds to determine the true meridian which constitutes the western boundary of the first range west and the eastern boundary

of the second range west of townships. Proceeding northward on the true meridian he sets quarter-section and section corners at 40 and 80 chains respectively and at 480 chains a township corner (N.W. cor. Twp.1N, R1W). He then runs due east on a random line a distance of 6 miles, setting quarter section and section corners at 40 and 80 chains to the east boundary of the range (in this particular case, the principal meridian) and notes the "falling" of the line. If this does not vary more than 3'00" of arc or more than 20 links per mile (80 chains)*, then this shall be established as the true line. But if the error in alignment or in distance exceeds such ratios, then from the "falling" he shall compute the true bearing, and shall, starting from the established point on the east boundary run back (westward on a true parallel) setting permanent quarter-section and section corners.

Arrived at the northwest corner of township 1 N, range 1 W, he proceeds in like manner to establish the remaining boundaries of township 2 N, range 1 W, by meridian, random line, and true parallel, and so with each township of range 1 west.

At the north township in each range any discrepancy, either excess or deficiency in measurement, shall be thrown in the last half of the section and the 40 chain station shall be greater or less than this distance by the amount of such discrepancy. So also the convergence of meridians shall be noted and closing corners set on the standard parallel north. In running the township

*These constitute errors of 1 in 4000 in measurement and 1 in 1800 in alignment.

boundaries of a 24 mile tract south of the base line, when for any reason it is deemed more expedient to run from north to south, the amount of the linear convergence shall be computed for the distance of 24 miles and the 6 miles set off on the base line, allowing for this convergence and the meridional line run as a true meridian intersecting the first standard parallel south exactly at the six mile point. The points set on the base line shall thus constitute the closing corners of the 24 mile tract next below the base line.

(147) The subdivision of the township into sections one mile square is as follows:

Each township is to consist of 36 sections surveyed and numbered as shown in Figure 54; section one in the northeast corner of each and every township, the numbering continuing to the west to include section 6; then sections 7 to 12 numbered toward the east; 13 to 18 toward the west, and so on alternately east and west, section 36 falling in the southeast corner of the township.

6	5	4	3	2	1
7	8	9	10	11	12
18	17	16	15	14	13
19	20	21	22	23	24
30	29	28	27	26	25
31	32	33	34	35	36

Figure 54

In beginning the subdivision of any township, the surveyor sets up the instrument at the southeast corner of the township, which is also the s.e. corner of section 36 of this township, and runs north on a true meridian

one mile, noting the location of the quarter-section and section corners previously established. He then returns to the southeast corner of the township and runs by a true parallel west as shown by the notes of the original survey of the township boundary observing the location of the quarter-section and section corners upon the parallel. This brings him to the southwest corner of section 36, which has already been monumented in the survey of the township exteriors previously made. From this point he runs north on a true meridian one mile and sets a permanent quarter section corner at 40 chains and a section corner at 80 chains north of the southwest corner. It is required that both the alignment and the measurement of these stations shall be made with extreme care and the line properly marked by blazing line-trees through wooded country; and proper monuments of a permanent character must be placed at quarter-section and section corners. Arrived at the northwest corner of section 36 the surveyor then proceeds east on a random line in a cardinal direction, sets a temporary quarter-section corner at 40 chains on this line, and at the distance of 80 chains notes the "falling" of the random with respect to the east boundary of the section and its northeast corner. If this exceeds 50 links (1/2 chain) more or less, he computes the true bearing of the line; starting at the northeast corner of the section, he corrects back on the line; he also corrects the measurement of the quarter-section (40 chains) distance and sets a permanent quarter-section corner in its true

location, and thence proceeds to the northwest corner of the section, number 36.

This is also the southwest corner of section 25 (see Figure 54). From here he proceeds to run the western boundary of section 25 on a true meridian, setting quarter-section and section corner; then by random to the cardinal east setting temporary quarter-section corner, correcting back on a computed true parallel, and establishing a permanent quarter-section corner as before and arriving at the northwest corner of section 25, which is also the southwest corner of section 24. And so he continues to the north tier of sections or to section one. Here if the measurement northward falls short or exceeds 40 chains on the last half of the measured western boundary of section one, then such excess or deficiency is placed in the last half of such line and the closing of the line on the north parallel boundary is marked with a closing corner and the distance to the nearest corner on the township north noted.

The subdivision is continued by returning to the south boundary of the township, and proceeding from here to the southwest corner of section 35 and as before running north on a true meridian, east on a random line, and correcting back as before; and so on until the last tier of sections to the west is reached. Here "having established the west and north boundaries of section 32, a random line will be initiated at the corner of sections 29, 30, 31, and 32, which will be projected westward parallel to the south boundary of the township setting a temporary quarter-section

corner at 40 chains, to an intersection with the west boundary of the township, where the falling will be measured and the bearing of the true line calculated, whereupon the line between sections 30 and 31 will be permanently marked between the section corners and the quarter-section corner thereon will be established at 40 chains from the east, thereby placing the fractional measurement in the west half mile as required by law"*.

The sections are not subdivided in the field by U.S. surveyors unless by special provision, but certain subdivisions-of-section lines are always projected upon the maps (official plats) and the local surveyor who may be employed by entrymen to run such lines in the field is compelled to correlate the conditions as found upon the ground with those shown upon the approved plat.

The unit of disposal under the general law is the quarter-quarter-section of 40 acres; the square mile (640 acres) is the unit of subdivision; while the unit of survey is the township of 36 sections (each one square mile).

(148) The function of the United States Surveyor has been fulfilled when he has properly executed and monumented his survey and returned an official record of it in the shape of complete detailed field notes and a plat. The function of the local surveyor begins when he is employed as an expert to identify the lands which have passed into private ownership; this may be a

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*This quotation and others to follow are from the Manual of Surveying Instructions, Survey of the Public Lands.

simple or most complex problem, depending largely upon the condition of the original monuments as affected principally by the lapse of time since the execution of the official survey. Also the local surveyor must when called upon be prepared to make the subdivisions of section, to discover the locations of "lost or obliterated" corners. To do this effectively requires a full knowledge of the methods of the survey, the rules applicable to these matters as laid down for the practice of surveyors, and the rulings of courts in certain cases. The subject of "lost and obliterated" corners, and also the judicial functions of the surveyor will be treated in subsequent assignments.

PROBLEMS:

1. A parcel of land is described as north half of the northeast quarter of Section 16 in Township 4 North, Range 3 East, Mt. Diablo Base and Meridian. Draw a map showing:

The initial point, base, meridian, twenty four mile parallel north, 24 mile guide meridian east; divide into townships, subdivide the townships above specified into sections and number them, locate the above described parcel in its proper place on the map. Make scale 1/4 inch = 1 mile. Designate each township by number and range.

Course 1A Elements of Surveying

Assignment 20

SURVEY OF THE PUBLIC LANDS (Cont.)

FOREWORD

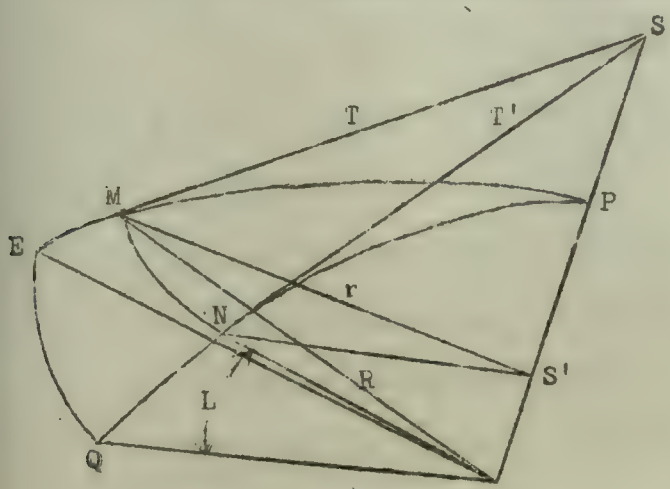
This assignment, continuing the general subject of "U. S. Public Land Survey", will explain Convergence of Meridians, the Tangent Method of running a parallel of Latitude, the Secant Method of running a parallel, and Obliterated or Lost Corners. For reference the student is directed to the Manual of Public Land Survey published by the General Land Office, Washington, D.C.

(149) CONVERGENCE OF MERIDIANS

As all meridians pass through the Pole of the earth, any two adjacent meridians are at maximum distance apart at the earth's equator and approach each other (converge) until they reach the pole where the distance between them is zero.

ANGULAR CONVERGENCE.

In Figure 55 let T and T' be the tangents to two given meridians whose angular convergence is required, MSN being that angle. Then if R is the mean radius of the earth and r that of the parallel of latitude MN (mean parallel of latitude under consideration) we have by spherical trigonometry



Angular Convergence
Figure 55

$r = R \cdot \cos \text{Lat.}$

$T = R \cdot \cot \text{Lat.}$

But by radian measure the arc MN subtends angles inversely proportional to the radii to this arc and since MN (in circular units) is the difference in longitude of the two meridians we may write

$$\frac{MSN}{\text{Diff. in Long.}} = \frac{r}{T}; \text{ hence } \frac{MSN}{MN} = \frac{R \cdot \cos \text{Lat.}}{R \cdot \cot \text{Lat.}}$$

$\frac{\cos}{\cot} = \sin$ (trigonometric relation), therefore,

$\frac{MSN}{MN} = \sin \text{Lat.}$, and $MSN = MN \cdot \sin \text{Lat.}$

In other words, the angle of convergence of two meridians is found by multiplying the difference in longitude by the sine of the mean latitude.

If the longitude is given in hours, minutes, and seconds, this value should be reduced to degrees, minutes, and seconds; and if the longitude is measured in linear units, this should be reduced to angular measure by finding the value of one degree using a value for r (the radius of the parallel arc) from the equation $r = R \cdot \cos \text{Lat.}$ While this is not precisely correct, the mean value R is sufficiently near for purposes of land surveying.

(150) LINEAR CONVERGENCE

In practice the linear convergence is set off in feet or chains. This may be determined from the formula $C/l = \sin MSN$, in which C is the linear convergence l is the distance on the meridian, and MSN is the angle of convergence. This equation

reduced is $C = l \cdot \sin MSN$.

As the meridians approach each other toward the pole, the correction is subtractive northward, additive southward.

A principal meridian is always established in the field by astronomical means, i.e., it is a true meridian as nearly as human agencies can determine. Meridians to the east of the principal meridian converge toward the principal westward; likewise those to the west converge toward the principal eastward. Hence the closing corners in the east quadrangle fall west of standard corners in the next quadrangle north, and such closing corners in the west fall east of their respective standard corners.

The linear convergence when determined for any township length of 6 miles may be set off 1/6, 1/3, 1/2, 2/3, 5/6 at each successive section corner; this simplifies computations.

(151) TANGENT METHOD OF RUNNING A PARALLEL CURVE

If it is desired to set out a true latitude curve east (or west) of a given point the Tangent Method may be used as follows:

If a line be run due east (or west) from any given point on the earth's surface, such a line, which is the arc of a great circle, departs from a true parallel by an amount which is a function of the latitude. At the equator the departure is zero, the departure increasing as the latitude increases. This may be seen by observing that a so-called straight line east or west is the arc of a circle having the earth's center for arc center, while

parallels of latitude are the circles formed by planes parallel to the equatorial plane.

A line projected east (or west) from any point in a parallel of latitude north of the equator will depart more and more to the south as it is carried east (or west) of the point. It will be tangent therefore only at point of beginning. This straight line is, hence, called the tangent and is conveniently made use of in laying out the parallel curve.

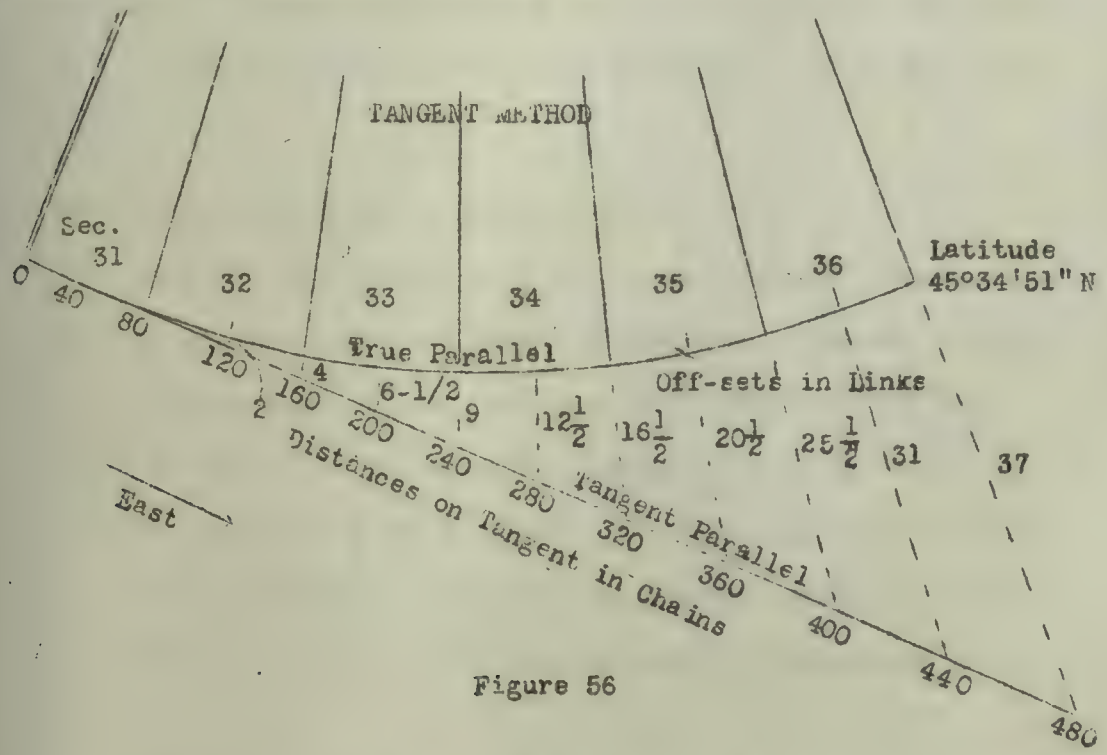


Figure 56

By reference to Figure 56 (exaggerated for sake of clearness) the method may be readily followed, thus:

Set up the transit and turn off an angle of 90° and project the tangent a distance of six miles, the measurements being made for each corner point at 40, 80, 120, etc. chains. Measure proper

offsets north from the tangent to the parallel, and upon the latter establish the corners. Standard Field Tables, prepared by the General Land Office and contained in the Manual, give the bearing angles and also the off-sets for each mile from 1 to 6. The half-mile offsets should be determined by interpolation; the quarter-section corners should be checked accordingly. The form of record is also given in the Manual by specimen field notes.

The tangent method is in favor in open or untimbered country. In a timbered country the blazing on the true line and the measurement of large off-sets are made with difficulty; hence the secant method is used.

(152) SECANT METHOD OF RUNNING A LATITUDE CURVE

This is a modification of the tangent method. Range out a line east (or west) for a distance of six miles, cutting the true parallel of latitude at the first and fifth mile corners and forming a tangent to an imaginary latitude curve at the middle mile point. Reference to Figure 57 will make plain the bearings and relative points of secant line and parallel curve, and the direction and order of the offsets required to accomplish the purpose.

A point on the meridian south of the township corner from which the parallel is to be run is located by computing, the distance being a function of the latitude. Then from the meridian the proper deflection angle is taken from table prepared by the General Land Office. Off-sets are measured and corners established

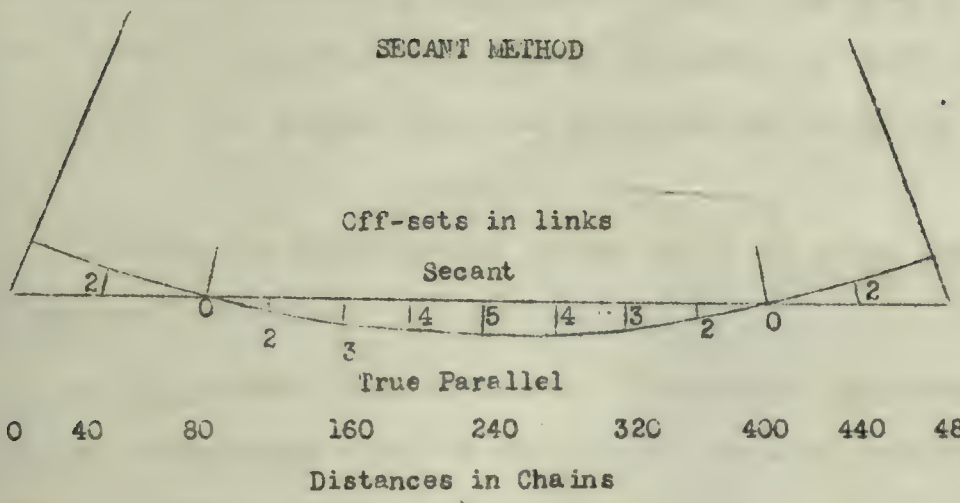
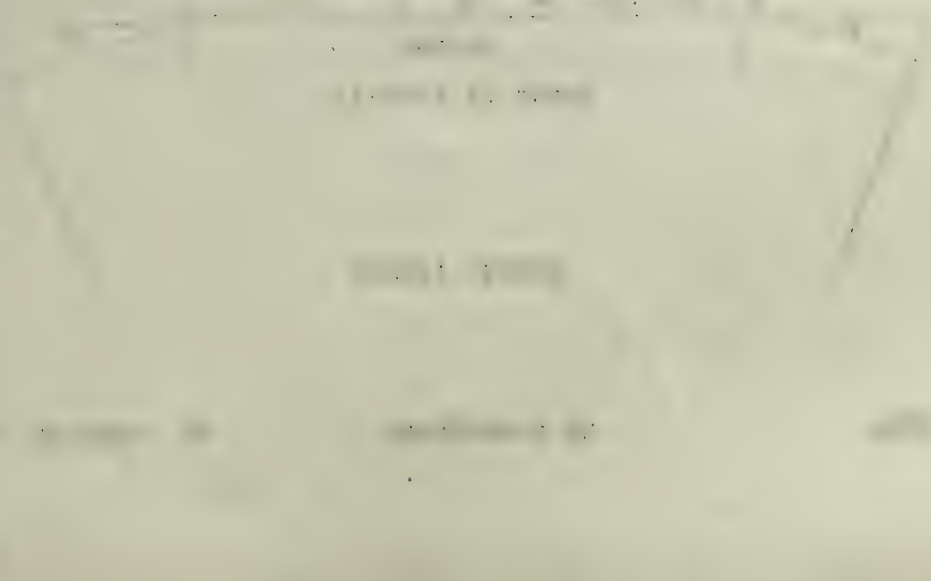


Figure 57

at every 40 and 80 chains; these also are taken from a table of off-sets. It will be noted that the distances south of the parallel on the meridional lines east and west are symmetrical, that the secant cuts the curve at the first and fifth mile points, that between these two points the parallel lies south of the secant and at the 0 and 6 mile points north of the secant. Furthermore, the off-sets are all short compared with most off-sets on a tangent and the secant, therefore, follows more nearly the line of the curve; this last fact renders the secant method in some respects preferable.

(153) RESTORATION OF LOST OR OBLITERATED CORNERS

Perhaps nothing in the work of the surveyor gives more trouble and therefore calls for greater care than the restoration of lost and obliterated corners. The best practice evolved through many years and regulated by the rules laid down by the General Land



Office and further defined by many court rulings is embodied in a chapter of the instructions contained in the Manual, to which you are referred. Some general rules and principles may be set forth here.

A distinction should be made in the use of the terms "corner" and "monument" as follows:

A corner means a point determined by the surveying process, while the term monument signifies the physical structure erected to mark the corner point upon the earth.

Again, distinction is made between lost and obliterated corners. If the physical evidence of location of a corner has been removed, but the marking can be restored upon adequate testimony or other available evidence the corner is not lost but is classed as obliterated. Where both the physical evidences and the immediate means of locating a corner are wanting, and lines must be run from other points and measurements by distance and angle taken to locate a missing corner, it is classed as a lost corner. A special procedure must, hence, be followed for the restoration.

Surveys once made and returned to the Commissioner by a Surveyor General and thus made of record in the Public Land Survey, are fixed and immutable for all time. Therefore, the surveyor has no authority to fix any point except by the means directed for that purpose. The surveyor then should note the difference between the regulations for the original survey of the

Public Lands and such as relate to identification of official surveys and the replacement of missing monuments on such surveys.

In determining whether a corner is lost or merely obliterated, the surveyor should examine into each case at every angle. The search for physical evidences having proved fruitless, he may take the corroborating testimony of witnesses, who can point out certain reliable evidences, as the intersections of walls, fences, center lines of roads, stumps of bearing trees, etc. Nor should he neglect to search for buried evidences, witness corners, and line trees as supplying reliable evidences of a valuable nature. When all such evidences fail the corner may be classed as a lost corner.

(154) METHODS OF RESTORING LOST CORNERS

A "single proportionate" measurement is one made in a single direction, east and west, or north and south, between two determinate points on the line. It consists in locating the corner in question a proportionate distance whether the true distance or not, by taking such proportionate measurement for the fractional distance as the whole measured distance is to the whole record distance.

When the lost corner is determined by measurements in two directions, the "double proportionate" method is employed. That is, proportionate measurements are instituted in both directions and should this method locate the corner both in latitude and longitude the same shall constitute the point required. If the measurements should not result in locating a point in common, then by cardinal off-sets, in latitude and departure, the corner shall be established.

If measurements are taken upon a principal meridian or a base line, or upon an established meridional line or parallel of latitude, the single proportionate method applies; but where the corner cannot fulfill these conditions, then measurements are taken in two, three, or four directions and double proportion must be resorted to.

Double proportionate measurement is generally applicable to the restoration of lost corners of four townships and lost interior corners of four sections.

Monuments north and south should control the latitudinal position of a lost corner, and monuments east and west should control the longitudinal position. Each identified original corner should be given a controlling weight inversely proportional to its distance from the lost corner.

Lost exterior section and quarter-section corners are restored by single proportionate measurements between the nearest identified corners on opposite sides of the missing corner, north and south on a meridional line, or east and west on a latitudinal line, after the township corners have been identified or relocated.

Lost interior quarter-section corners are to be restored by single proportionate measurements between the adjoining section corners, after the section corners have been identified or relocated. (Note the emphasis on "after".)

Lost meander corners, originally established on a line across a meanderable body of water and marked upon the opposite side of it will be relocated by a single proportionate measurement, after the section or quarter-section corners upon opposite sides of the missing meander corner have been identified or relocated.

There are other provisions for the relocation of corners and retracement of lines which are beyond the scope of these assignments, and the student must not presume to follow the meager directions here given. Here we give only the outline of controlling principles and it is not desired to cover these up with a large mass of details.

You are referred to the "Manual of Surveying Instructions" and to a brief treatise on "Restoration of Lost and Obliterated Corners" issued by the General Land Office, Washington, D. C.

PROBLEMS:

1. What is the angular convergence of meridians 6 miles apart in latitude 40°? The length of 1° of longitude in latitude 40° is 53.053 miles.

2. What is the linear convergence in links at 6 miles? 1, 2, 3, 4, 5, miles ?

3. The N.W. quarter of the N.W. quarter of a certain section measured 20.16 chains by 18.24 chains; what is the content in acres?

UNIVERSITY OF CALIFORNIA EXTENSION DIVISION

Correspondence Course

Course 1B Elements of Surveying Swafford

Assignment 21

STADIA SURVEYING

FOREWORD

This assignment is designed to explain the principle and method of the stadia, the scope and limitations of its use, and also the application of stadia measurements to the various branches of surveying - land, railroad, topographical, etc.

(156) The Stadia

It is a rod marked with linear units of measure which may be read afar off, and from the measures thus obtained heights and distances may be computed. The name "Telemeter" has been proposed and is occasionally used for stadia; this name (tele = afar, meter = measure) like telescope, telegraph, telephone, indeed fits the thing and describes its use, but usages change slowly and stadia still persists in the language of surveying.

(157) Telescope and Stadia - Hairs

To aid in reading the distant rod a telescope is used. This gives large range and great definiteness to the distant markings and characters. Also two fine wires or spider-webs are introduced at the principal focus of the telescope and these are spaced at such a distance apart that they intercept a given interval on the rod at any given distance from the center of the telescope. It is common practice, and for purposes of computation quite convenient, to space these stadia wires (stadia-hairs) so that an

interval of 1 foot is intercepted on the image of the rod when the latter is 100 feet from the instrument; by the law for similar triangles, at 200 feet the interval would be 2 feet; at 300 feet, 3 feet; at 1000 feet, 10 feet; etc.; the law holding, of course, for all distances small or great.

Thus, by observing through a telescope furnished with these stadia-hairs, directed upon a rod held at some distant point, and by knowing the ratio of the distance to the rod intercept, the distance of the rod from a point near the observer may be computed.

(158) Theory of Lenses

Before proceeding further with the subject of this assignment, it is best to explain briefly the phenomena of light passing through the lenses of a telescope, as follows:

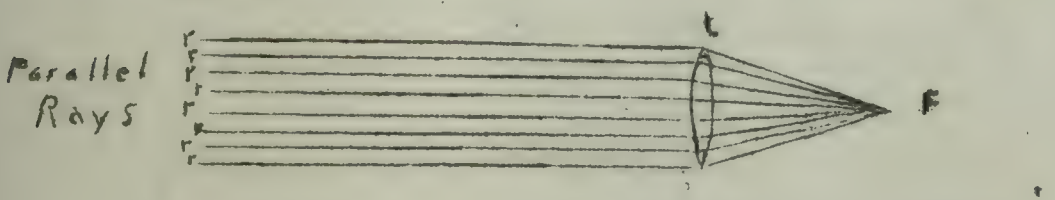


Figure 58

rrr r r r r are parallel rays of light (the rays from the sun, or a star or of other very distant luminous point are practically parallel). Such rays passing through the double convex lens, L, are refracted toward the axis of the lens upon their passage, and are again refracted in passing through the air, and converge at F, the principal focus.

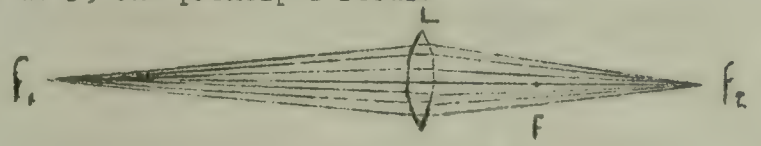


Figure 59

f_1 and f_2 are conjugate foci; that is, light rays emanating from f_1 are focused at f_2, and vice versa.

If f_1 is the measured distance from L to the focus in the left, f_2 the distance of its conjugate focus, and F the distance of the principal focus, then by the law of optics, $\frac{1}{f_1} + \frac{1}{f_2} = \frac{1}{F}$

In the following figure, let AB be the rod intercept as seen through the telescope and included between the stadia-hairs, ab the image (inverted as shown), i the stadia-hair interval, and f_1 and f_2 the distance of image and rod respectively; then by geometry:

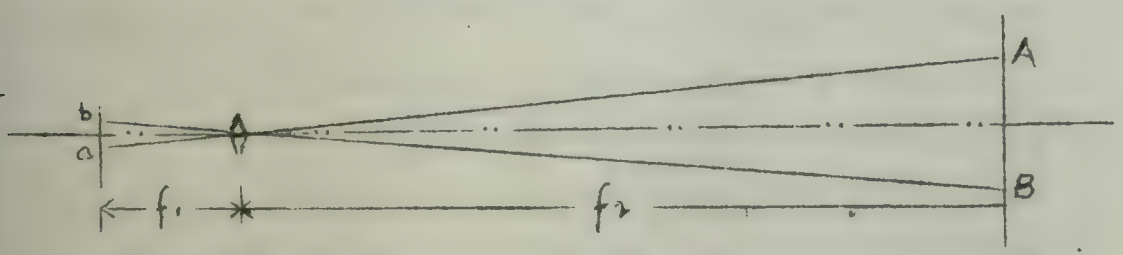


Figure 60

$f_2 : f_1 :: AB : ab$. Putting S for AB (the rod interval), i for ab, the stadia-hair interval and writing in fractional form, we have $\frac{f_2}{f_1} = \frac{S}{i}$.

(159) Principles of Stadia Method

These are two: the first being that of the law of proportion for similar triangles, which is purely geometric. The second is the law for conjugate foci of double convex lenses.

1) When light from an infinitely distant source falls upon a convex lens the rays are parallel and are focused on the opposite

side at a point called the principal focus, F. If the light emanates from a near (or finite) source, f_2, the rays are divergent and, by the law of refraction, are in such cases brought to a focus, f_1, that falls beyond the principal focus. Again, the point f_1 may be considered as the radiant point and f_2 the focus. Hence, f_1 and f_2 are called conjugate foci. The law of optics above alluded to may now be stated as follows:

$$\frac{1}{f_1} + \frac{1}{f_2} = \frac{1}{F} \qquad (1)$$

Or in words thus: The reciprocal of the principal focal distance (F) is equal to the sum of the reciprocals of the distance of the conjugate foci, all measured from the optical center of the lens.

2) Suppose now that an interval upon the stadia-rod is imaged between the two stadia-hairs situated at the rod's conjugate focus. Lines drawn through the optical center of the lens, not being refracted, are therefore straight lines, the ray from the upper end of the rod interval A will fall at a on the lower cross-hair (image inverted). AB = rod interval, ab . stadia-hair interval.

Now call the rod interval S, the stadia-hair interval i, and the distances of the conjugate foci, f_1 and f_2 as before, and by geometry we have:

$$\frac{f_2}{f_1} = \frac{s}{i} \qquad (2)$$

Solve the simultaneous equations (1) and (2) for f_2, i.e. the distance of the rod from the objective end of the telescope, and we

find:

$$f_2 = \frac{F}{i} s + F \qquad (3)$$

Thus it is seen that the distance from the objective to the rod is made up of two parts, $\frac{F}{i} s$, which is variable depending upon the rod interval, and the focal distance, F, which is constant for any given instrument. Moreover, it is always desirable to measure the distance of the rod not from the objective but from the center of instrument (i.e. from the vertical axis). It is, therefore, necessary to add another quantity which we will call c, the distance of the center of the instrument from the objective. Hence we combine the quantities F and c, giving a value, F + c, which is regarded as a constant for a given instrument, and has a range in value of from 0.75 ft. to 1.50 ft., usually regarded approximately one foot.

The distance c in instruments that have the adjustable objective, is variable, but the error introduced thus is a negligible quantity. It may always be regarded as constant, and F+c determined for any instrument is, as also $\frac{F}{i}$, in instruments having fixed stadia-hairs, and, the principal focal length of the objective, being constant, $\frac{F}{i}$ is a constant ratio.

The ratio $\frac{F}{i}$ is determined by the maker, and the interval between the stadia-hairs fixed accordingly. In some instruments the interval i is made adjustable and may then be set by the user to suit either a given rod or a desired ratio, $\frac{F}{i}$.

No.	Distance	U.C.W.	L.C.W.	R	E_t	E_t
1	[illegible]	[illegible]	[illegible]	[illegible]	179.24	99.02
2	268.40	5.86	3.16	2.70	267.23	98.96
3	355.32	6.29	2.71	3.58	354.15	98.92
4	400.89	6.02	1.97	4.05	399.72	98.72
5	482.80	7.03	2.17	4.86	481.63	99.11
6	556.30	7.80	2.19	5.61	555.13	99.20
7	643.58	7.25	0.75	6.50	642.41	98.84
8	786.93	8.45	0.55	7.90	785.76	99.47
9	914.40	9.77	0.42	9.15	913.23	98.91
10	1024.71	10.55	0.24	10.31	1023.54	99.26
					10	990.41
					Mean E_t	99.04

c = 0.47 ft.
F = 0.70 ft.
F+c = 1.17 ft.

Q.M.L. Transit 11.76
Target Rod #16
Tape #1R
Chaining Pins

Cloud, T.
Harris, J.
Morse, T.
Jones, L. Recorder

Set 10 chaining pins in line at approximate 100-foot intervals.
With transit telescope level, determined the readings for U.C.W. and L.C.W. when rod was held at each pin. Computed R.
Measured distance from center of instrument to each pin with steel tape - tape standardized - to nearest 0.01 foot.
With objective focused on a distant object determined F by measuring from center of objective to crosshairs.
With objective focused on an object about 300 Feet away determined c by measuring from center of objective to center of instrument.
Determined different values of E_t from formula

$$H.D. = E_t R + (F+c)$$

Example:- For point #1

$$H.D. = 180.41 = (E_t \times 1.81) + (1.17)$$

$$E_t = \frac{180.41 - 1.17}{1.81} = 99.02$$

NOTE:- Targets read to nearest 0.01 Foot.

Although the ratio $\frac{F}{i}$ is usually fixed by the maker and the quantity F+c depends upon the construction of the optical parts of the telescope, it is best, always, to determine these constants for any given instrument, and the student is urged to "try out" the instrument to that end, in the manner shown as follows:

(160) To determine the constant F+c

With an engineer's scale measure the distance from the middle of the objective lens to the reticule that carries the stadia-hairs, when the instrument is focused on a very distant object, such as a star, the sun, or some other very remote point. By so focusing the instrument, the rays of light entering the objective are made practically parallel and, therefore, the image is at the principal focus. This measured distance is F. Now with the instrument focused upon some object at a mean distance of observation (which is ordinarily about 500 feet from the observer), measure the distance from the middle of the objective to the vertical axis of the instrument (i.e. the center of the telescope, which is the horizontal axis of the same). This gives C. The sum of these two distances, F+C, is the constant required.

(161) To determine the ratio $\frac{F}{i}$

Set up the transit on a level stretch of 600 to 1000 feet. By means of markers placed at varying intervals establish ten or more points in a straight and level line. Level the telescope so that the line of sight may be truly horizontal. With the leveling rod and two targets obtain rod intervals for each distance set off

by the markers (pins), setting the lower target on the lower cross-hair and the upper target on the upper cross-hair (for each distance in all cases the rod must be plumb). Measure the distance of each pin from the transit point; deduct the constant F+c, previously determined, as above, for the measured distance in each case (or better, set off the distance F+c in front of the transit point and take all measures of distance from this forward point). The rod interval (i.e. the distance between the targets) in each case divided into the measured distance will give the ratio $\frac{F}{i}$. The average of all these $\frac{F}{i}$'s is the mean constant of the instrument. CAUTION: Should any one or more of the determinations differ widely from the general observed value, such may be considered in error and of course rejected in obtaining the nearest mean, $\frac{F}{i}$.

For further explanation of these methods see the numerical examples at the end of the assignment.

(162) Inclined Sights

Our treatment of the matter of stadia measurement so far has been confined to such as may be made on level ground, in which case the horizontal distance is measured direct. But it is seldom that this condition obtains; sights are taken to points above or below the level of the instrument. Such are called inclined sights, and the distances so measured must be reduced to the horizontal for purposes of surveying and mapping. If the rod could be inclined over the point of observation so that the line of sight was perpendicular to the rod at that point, the distance obtained thus from the rod-reading (stadia interval) would express the slope

distance. The horizontal distance, in that case, would be the slope-distance multiplied into the cosine of the angle of elevation (or depression). The vertical height of the point (over which the rod is held) would also be the slope-distance into the sine of the angle of elevation (or depression).

In the following figure, the horizontal distance H.D. is required.

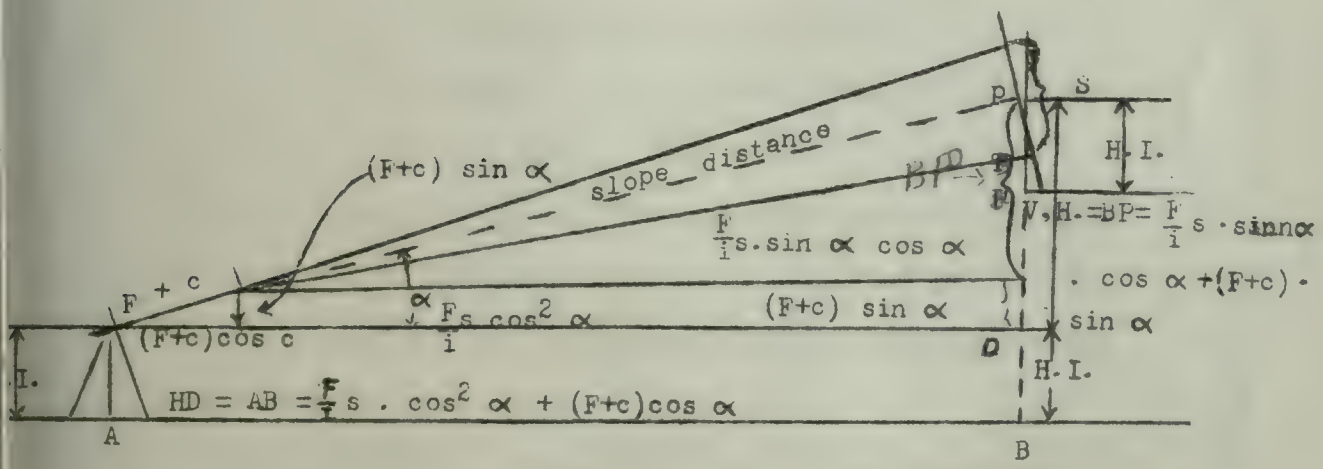


Figure 61

Here S is the rod-reading, when the line of sight is directed to the point p on the rod held vertically above the point P on the ground, the rod being inclined toward the transit, so that the line of sight (i.e. line of collimation of the telescope) will be perpendicular to the rod; this point p is also at a distance above P, equal to the height of the instrument, H.I. The slope-distance measured along the line of sight from the center of the instrument to p on the rod is $\frac{F}{i}s + (F+c)$ and the horizontal distance, A.B.,

is evidently the slope-distance times the cosine of the angle of elevation α

Expressed in symbols thus:

$$A\,B = H.D. = \left[\frac{F}{i}s + (F+c)\right] \cos\alpha \quad \text{---} \quad (1)$$

Likewise the difference in elevation of the point P over transit point A is:

$$B\,P = V.H. = \left[\frac{F}{i}s + (F+c)\right] \sin\alpha \qquad (2)$$

Having the horizontal distance, we may find the difference in elevation simply by multiplying by the tangent:

$$V.H. = H.D. \tan\alpha = \left(\frac{F}{i}s + (F+c)\right) \cos\alpha \cdot \tan\alpha$$

(By trigonometry $\sin\alpha = \cos\alpha \cdot \tan\alpha$)

Of course in all cases where distances are taken on level (horizontal) sights the simple formula, $\frac{F}{i}s + (F+c)$ is very convenient and should be used, both because it minimizes liability to error and because it simplifies computation.

Also, it has been proposed (and indeed some instruments have been so constructed) that the stadia-hairs be placed vertical to read upon a rod held horizontal instead of being placed horizontal to read on a rod held upright but perpendicular to the line of sight. But both these methods are not always convenient in practice and the usual way is to hold the rod vertical over the point and apply the necessary reductions for obtaining the horizontal distance, H.D. and vertical heights, V.H. This calls for a further consideration of the method known as:

163) Inclined Sights with Rod Vertical

In the following figure let AB be the horizontal distance,

s equal the rod-reading (on rod held vertical) AC = BD = height of instrument (H.I.), which is also equal to the distance of the middle cross-hair above the point P; hence PB = DO = V.H. Likewise F+c, as shown, may be reduced to (F+c) cos α, or (F+c) sin α as required for H.D. or V.H.

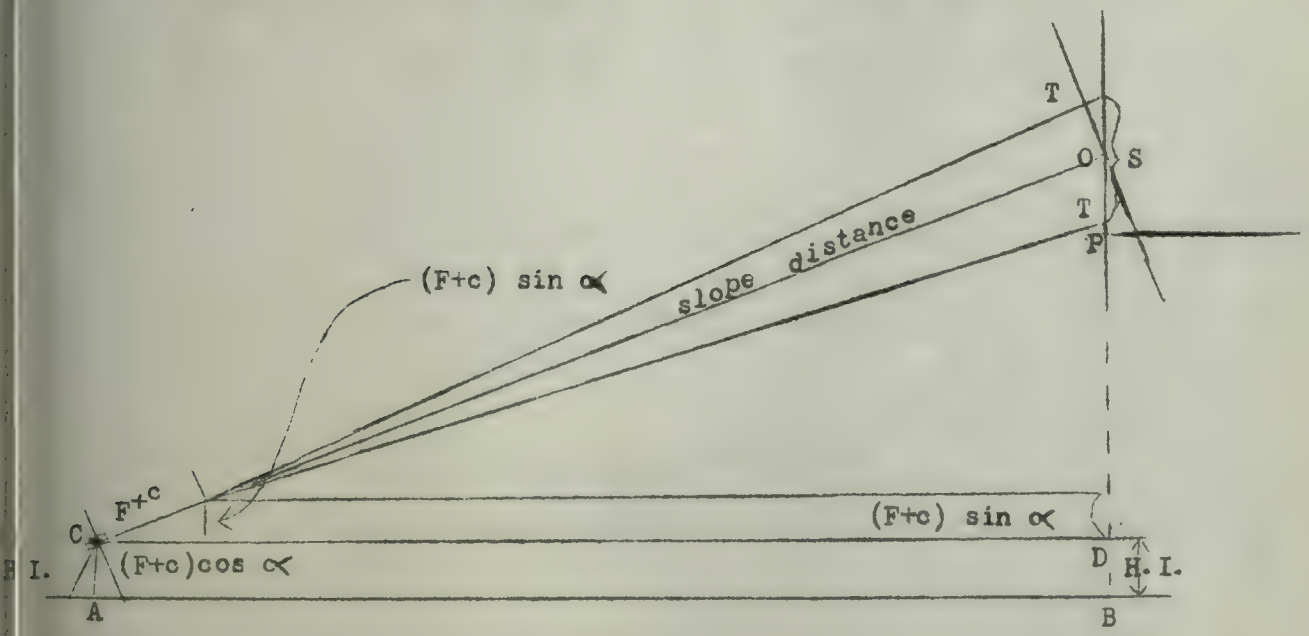


Figure 62.

It can be shown that the horizontal distance AB = H.D. =

$$\frac{F}{i}s \cdot \cos^2 \alpha + (F+c) \cos \alpha \qquad (3)$$

and that the difference in elevation

$$BP = V.H. = \frac{F}{i} s \cdot \cos \alpha \cdot \sin \alpha + (F+c) \sin \alpha \qquad (4)$$

Thus two important formulas, for the determination of the two desired quantities, H.D. and V.H. are here explicitly set forth, and you should become familiar with them, for frequent reference will be made to them in further considerations and in computations.

Certain changes are made in these formulas, 3 and 4, for the purpose of simplifying computation, and also for the construction of tables suitable for stadia reduction. If we examine the products (F+c) sin α and (F+c) cos α and compare these with the values (F+c) sinαcosα and (F+c) cos$^2\alpha$, we find that, in general, the difference in value is negligible. Hence the term (F+c) may be added to the inclined distance, $\frac{F}{i}s$, and these formulas become

$$H.D. = \left[\frac{F}{i}s + (F+c)\right] \cos^2 \alpha \qquad (5)$$

$$V.H. = \left[\frac{F}{i}s + (F+c)\right] \cos \alpha \cdot \sin \alpha \qquad (6)$$

which are approximate formulae, and are commonly employed. Therefore briefly -

(164) RULES for Stadia Reduction with Inclined Sights

For Horizontal Distance multiply the rod-reading by 100*, add (F+c), and multiply by the cos^2 of the angle of elevation, α.

For Vertical Height, multiply the rod-reading by 100*, add (F+c), and multiply by both the cos and sin of the angle of elevation.

N.B.Remark. Having determined the horizontal distance, it is only necessary to multiply this quantity by the tangent of the angle of elevation to get V.H.

(165) Stadia Reduction Tables

Tables will be found in the manuals and texts usually referred to - Johnson, Breed and Hosmer, Tracy and Raymond, with adequate explanation for their use. Logarithmic tables for this

---

*This is the common ratio F/i; if the constant for the instrument differs from this, use the correct value for F/i of the given instrument.

PROB. NO. 14. AZIMUTH TRAVERSE WITH TRANSIT AND STADIA

Station Inst.	Station Obj.	Azimuth	Magnetic Bearing	U.C.W.	L.C.W.	Rod Interval	Vert. Angle	Horizontal Distance
A	D		S 55° 00' E	6.897	3.104	2.793	-4° 10'	
	B	185° 00'	N 5° 00' E	5.378	3.722	1.756	+5° 20'	178.8
B	A		S 5° 00' W	5.378	3.721	1.755	-5° 20'	
	C	305° 00'	S 54° 55' E	6.897	3.104	2.793	-4° 10'	284.6
C	B		N 55° 00' W	6.898	3.104	2.794	+4° 10'	
	D	5° 00'	N 54° 54' W	5.378	3.722	1.756	-5° 20'	178.8
D	C		N 4° 55' E	5.378	3.721	1.755	+5° 20'	
	A	125° 01'	N 55° 00' W	6.897	3.104	2.793	+4° 10'	284.6

COMPUTATIONS

		AB	BC	CD	DA
log $\frac{f}{f}$		2.00860	2.00860	2.00860	2.00860
log R+	$\frac{ft}{mm}$	0.24748	0.44793	0.24748	0.44793
log H.D (table)		9.99623	9.99770	9.99623	9.99770
log H.D (comp.)		2.25231	2.45423	2.25231	2.45423
H.D		178.8	284.6	178.8	284.6

INTERIOR ANGLES

A = 180° 00' - (185° 00' - 125° 00') = 120° 00'

B = 180° 00' - (305° 00' - 185° 00') = 60° 00'

C = 180° 00' - (365° 00' - 305° 00') = 120° 00'

D = 180° 00' - (125° 00' - 5° 00') = 60° 00'

Check 360° 00'

Equipment:
K + E. Transit # 10965
Target Rod # 4.5
Plumb-bob
Reading Glass
Pocket Tape

Dec. 13, 1918.
Cool + cloudy.

Johnson, J. E.
Wells, C. M.
Shafer, A.

Notes.

NOTE:

$\frac{f}{i}$ = 102.0 (Prc) = 1.80

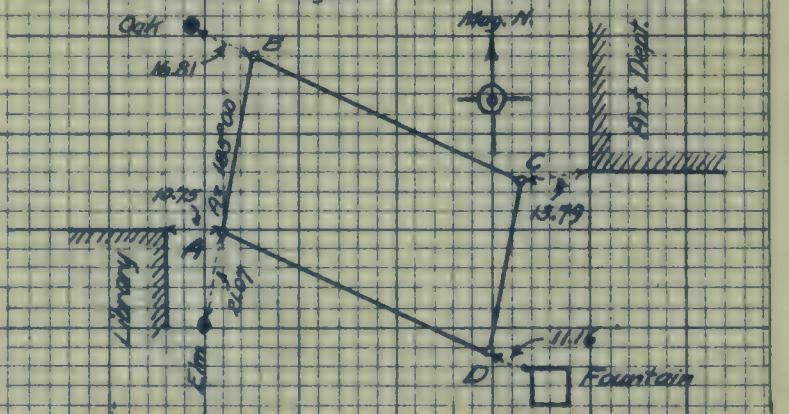
Used magnetic south as zero azimuth.

Sighted M.C.W. on H.I. for the determination of vertical angles and rod intervals.

Took forward and backward readings at each station.

Used value of forward reading in computations where readings did not vary more than 0.001 foot.

Determined Horizontal Distances and Interior Angles.



use have been compiled and we would especially call attention to those of Prof. F. S. Foote, Jr., University of California Publications in Engineering, Vol.I, No. 11. These tables, especially the last for those familiar with logarithmic methods of computation, reduce the work to a minimum and save the computer both time and labor.

Note. Among the Problems to accompany this assignment, two illustrative examples are worked out in detail to show the methods of computing by use of the exact formula in one case, by the approximate formula in the other.

The uses of the Stadia Method in land, topographical, and railroad surveying, will be explained in the following assignment.

References:

Tracy, pp. 303 - 308

Raymond, pp. 127 - 140

Johnson, pp. 224 - 233

Breed and Hosmer, pp. 188 - 201, Vol. I.

The following data, arranged in tabular form furnish the elements of problems in finding the horizontal distance and vertical distance from observed rod-reading and vertical angle.

Transit at M F/i = 100.2; f+c = 1.2

Sta.	Azimuth	Rod	Vert. Ang.	F/i S + (f+c)	H. D.	V. D.
A	107°40'	6.73	3°24'	675.5	673.1	40.0
B	241°15'	8.67	16°30'	869.9	799.7	236.9

COMPUTATIONS FOR STATION A:

Approximate Method

F/i S + (f+c) = 6.73 x 100.2 + 1.2
= 674.3 + 1.2
= 675.5

673.1 x tan 3° 24' =
673.1 x .0594 = 39.98
V. D. = H. D. x tan

H. D.
log F/i S + (f+c) = 2.82963
" cos 3°24' = 9.99923
" " 3°24' = 9.99923
2.82809

H. D. = 673.1 = Antilog.

V.D.
log f/i S + (f+c) = 2.82963
" cos 3° 24' = 9.99923
" sin 3° 24' = 8.77310
1.60196

V. D. = 39.99
40.0 = Antilog.

Exact Method

F/i S $\cos^2\alpha$

log F/i (100.2) = 2.00087
" S (6.73) = 0.82802
log cosα(3°24') = 9.99923
" " " (3°24') = 9.99923
2.82735

672.0 = Antilog

672.0
1.2
H. D. = 673.2

(f+c) Cos α

log (f+c) (1.2) = 0.07918
" cosα(3°24') = 9.99923
0.07841

1.2 = Antilog

F/i S cos α Sin α *

log F/i (100.2) = 2.00087
" S (6.73) = 0.82802
" cosα(3°24') = 9.99923
" sinα(3°24') = 8.77310
1.60122

39.92 = Antilog

(f+c) sin α

log (f+c) (1.2) = 0.07918
" sinα(3°24') = 8.77310
8.85228

0.07 = Antilog

39.92
0.07
39.99

V.D. = 40.0 ft.

Thus for small vertical angle the V. D. is the same by the approximate and the exact methods. The H. D. by the two methods differs by 1/10th of a foot, which is within the limits of accuracy for mapping.

COMPUTATIONS FOR STATION B

Approx. Method

F/i S + (f+c) = 8.67 x 100.2 + 1.2
= 868.7 + 1.2
= 869.9

V.D. = H.D. x tan α
= 799.7 x .2962
= 236.87

H. D.

log F/iS + (f+c)	= 2.93947
" cos α (16°30')	= 9.98174
" " " "	= 9.98174
	2.90295

H. D. = 799.7 = Antilog.

V.D.

log F/iS + (f+c)	= 2.93947
" cos (16°30')	= 9.98174
" sin (16°30')	= 9.45334
	2.37455

V. D. = 236.9 = Antilog

Exact Method

F/i S · $\cos^2 \alpha$

log F/i (100.2)	= 2.00087
" S (8.67)	= 0.93802
" cos α (16°30')	= 9.98174
" " α "	= 9.98174
	2.90237

798.7 = Antilog

(f + c) · Cos α

log (f+c) (1.2)	= 0.07918
" cos α (16°30')	= 9.98174
	0.06092

1.1 = Antilog

798.7
1.1
H. D. = 799.8

F/i S·cos α· sin α

log F/i (100.2)	= 2.00087
" S (8.67)	= 0.93802
" cos α (16°30')	= 9.98174
" sin " " "	= 9.45334
	2.37397

236.6 = Antilog

(f+c)· sin α

log (f+c) (1.2)	= 0.07918
" sin α (16°30')	= 9.45334
	9.53252

0.34 = Antilog

236.6
0.3
236.9

* The expression cos $\propto$. sin $\propto$ is equivalent to 1/2 sin 2 $\propto$, and is commonly put in the latter form. When using natural functions in computation, this latter form is of advantage as it reduces operations to a minimum. But in logarithmic computation the log cos and log sin are found together in the table and it is therefore more convenient to use these than to turn to the part of the tables giving sin 2 $\propto$ and then also to be obliged to join it with the colog. 2 (log 1/2).

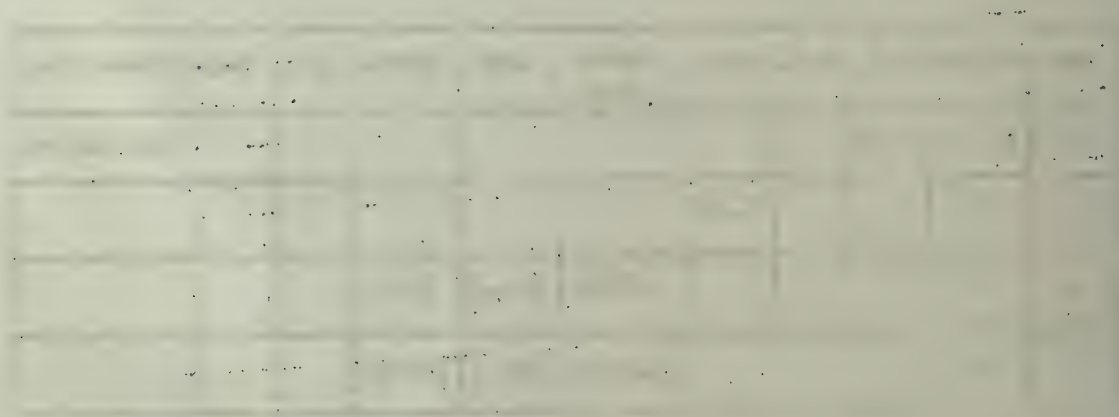
P R O B L E M

Given the following data:

$\frac{E}{i}$ = 102.4, F + C = 1.12, B.M. 107.54 ft.

Sta.	Sight	B.S.	H.I.	F.S.	North Azim.	Rod	Vert	H.D.	V.H.	Elevation
X	B. M.	4.89								107.54 Ft.
	A			6.83						
A	B				17°23'	234.5	7°50'			
	C				152°17'	452.7	10°35'			

Determine the horizontal distance, A -B, B - C, C - A; also Vertical Height above A, of B, C, and the elevation above datum of A, B, and C. Also find the interior angles of the triangle ABC, and the azimuth (from the North) of B - C. Sketch the triangle and place the above determined values appropriately upon the sketch.



UNIVERSITY OF CALIFORNIA EXTENSION DIVISION

Correspondence Course

Course 1B Elements of Surveying Swafford

Assignment 22

STADIA SURVEYING - STADIA REDUCTIONS

Foreword:-

This assignment will treat of the method of the stadia as applied to land, railroad, and topographic surveying, and the use of stadia reduction tables and diagrams.

(166) In General.

The stadia method of measuring distances, both horizontal and in elevation, is applicable within certain limitations, to all kinds of surveying and furnishes a rapid and convenient means of accomplishing the work. In open country where lines need not be cleared of growth that obstructs the line of sight, two operators only are required; however, an alert transitman may keep two or several rod-men busy selecting points and giving rod. If the selection of points, as in topographic details, or for locating contours, is to be made by the rod-man, he should be intelligent, quick to discern the salient features sought, and fully versed in the work in hand. Otherwise he must act wholly under direction of the instrument man, who designates the points to be occupied by the rodman as the work progresses. Where the survey is over a specified traverse previously determined or staked off, the rodman goes from point to point.

(167) Stadia Rods

These vary in style and scale of graduation in accordance with the work in hand. The ordinary level-rod is convenient where readings are desired in feet and fractions; and for long sights, a target (or better two targets) may be employed to advantage. If the constants of the instrument, $\frac{F}{i}$ and f+c, have been carefully determined, there is little difficulty in making computations for H.D.'s and V.D.'s from readings taken with the ordinary level-rod.

(168) Speaking Rods

These are especially convenient over short distances ranging up to 1000 feet and are carefully designed and marked to assist in quick and accurate reading through the telescope. Such rods may be graduated in feet and decimals; the tenths being explicitly marked and the hundredths being read by estimation. The whole foot mark and the 0.5 ft. division are also specially indicated to facilitate reading and to avoid confusing these points. Furthermore, for like reasons, the numbering of the full foot-marks is also designed to avoid mistakes, so the 3 is made of the shape here given, rather than thus, ℨ, which at long range may be mistaken for 8; a Roman V is employed instead of 5, and X for 10. Caution should be exercised in reading 6 for 9, or vice versa, when sighting with an inverting telescope. In the last case, it is best to observe whether the number read is near the V or the X; if the former, it is 6; if the latter, it is 9. The illustration on the following page shows a good form of speaking rod. These may be had from manufacturers in a great variety of designs; or the engineer may devise a pattern for his own use to suit his own ideas.

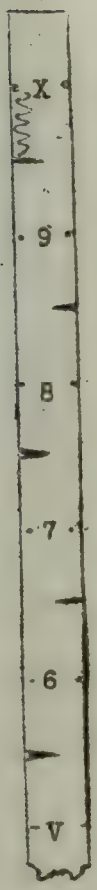


Fig. 63

(169) Stadia Rods are sometimes graduated to suit a given stadia-hair interval; this may be accomplished by staking off a stadia course on level ground with stations at 100, 200, etc., up to 1000 feet, and locating the rod intervals as viewed through the telescope by the fine pencil lines drawn upon the rod. Note that F + C should be carefully determined and set off from the plumb-line of the transit to the zero station of the course. In general it is better to employ a rod graduated in feet and decimals, as such a rod may be used with any telescope of which the constant $\frac{F}{I}$ has been accurately determined.

(170) For reading rod intervals at long distances or when, for any reason, the whole interval is not visible or distinct, the semi-interval may be observed; twice such a reading is then the rod-reading. This requires that a middle cross-hair, (such as found in most instruments) shall be exactly midway between the upper and lower stadia-hairs. To avoid mistakes in reading the rod, it is sometimes expedient to read both the semi-intervals and check the sum of such double readings also by observing the full interval. At another time the lower interval may not read correctly owing to unequal atmospheric refraction in the lower strata of the air. Then the upper interval should be relied upon as the more nearly correct means.

(171) Surveying by stadia over country broken by hills and ravines is not only expeditious, but usually is more precise than measuring

distances by chain or tape. When greater precision is required than can be secured by stadia methods, resort may be had to the more accurate method by triangulation for principal lines and for control, and minor details requiring less degree of accuracy can then be filled in by stadia.

Very long lines greater than the range of the instrument, say lines of 1500 to 2000 feet in length, may conveniently be measured by "balancing in" the transit at an intermediate point in the line and sighting the rod in both directions; in other words, by measuring the line in two segments. When doing this the constant F + c enters into both segments. The device of "balancing in" in this manner is especially applicable in case an intervening hill prevents viewing the further point; on such an occasion station the transit upon the elevated position on line and take readings as before in both directions.

(172) Traversing by Stadia.

In this work it must be remembered that three things are necessary to be done. (1) The angular relation of the lines in a horizontal plane must be determined; (2) The rod-reading must be taken; (3) The angle of elevation must be measured. And a fourth - the magnetic bearing is valuable as a check and it is recommended that it should be observed to that end, occasionally, at least.

Of course, any of the methods of measuring horizontal angles with their appropriate checks may be used, but the method by azimuths is generally preferable; this is especially true when mapping is to follow, particularly topographic mapping with many details.

It is important that the angle of elevation be correctly determined; hence the line of sight of the telescope must be directed to a point on the rod as much above the ground point as the horizontal axis of the telescope is above the point on ground over which the transit is set. This distance is called H. I. (height of instrument) and is obtained by actually measuring the interval from the ground to the center of the trunnion of the horizontal axis. For this purpose the instrument man usually carries a short tape and measures the H.I. at each setting. Some require that the rodman shall take this interval by means of the rod set up next the instrument before going to the point to be observed upon and then setting a target or other marker at this height upon the rod. However, the tape is more convenient and obviates the necessity of the rodman running to and fro for this purpose. A rubber band or string passed around the rod at H. I. serves well in place of a target; on speaking-rods the target is wanting.

The best way for sighting the rod is to bisect the H. I. on the rod with the middle cross-hair; then set the lower stadia hair on the nearest full foot-mark; read the rod interval as given by both stadia-hairs, if possible; then, having recorded the rod reading, again bisect the H. I. with the middle cross-hair and, while the rod-man is going to the next point, take the reading of the vertical arc, i.e. read and record the vertical angle. Also read and record the horizontal angle.

(The deflection angle, azimuth, or bearing, as the case requires.) The needle having been lowered, also observe

PROB. NO 15. TOPOGRAPHY WITH TRANSIT AND STADIA.

Sta.	Obj.	Azimuth	Vert. Angle	Rod Intercept	Hor. Dist.	Diff. Elev.	Elevation	Remarks
A	I	0°00'	- 2°10'	3.41	345	-13.0	102.8	Point in saddle S.E. of A.
	IV	108°12'	- 4°14'	2.11	213	-15.8	100.0	On brow of hill N.W. of A.
	1	7°40'	0°00'	2.07	210	0.0	115.8	Top of knoll S. of A.
	2	9°12'	- 3°04'	3.05	301	-16.1	99.7	End of ridge S. of A.
	3	13°38'	- 2°12'	1.76	178	- 6.9	108.9	Foot of slope W. of Sta 1.
	4	17°14'	- 6°10'	0.54	55	- 5.9	109.9	Head of broad draw S. of A.
	5	25°59'	- 4°16'	0.97	98	- 7.3	108.5	Bend in fence S. of A.
	6	42°42'	- 2°00'	1.96	199	- 6.9	108.9	Break of ridge S.W. of A.
	7	72°58'	- 3°02'	2.51	254	-13.4	102.4	Bend in fence S.W. of A.
	8	85°00'	- 2°16'	3.02	306	- 12.1	103.7	Low point in saddle W. of A.
	9	93°11'	- 1°06'	4.75	480	- 9.2	106.6	Point on fence at end of ridge.
	10	106°44'	- 2°02'	1.53	155	- 5.5	110.3	Point on ridge N.W. of A.
	11	119°10'	- 6°08'	5.09	509	-54.7	51.1	Point in draw N.W. of A.
	12	145°10'	- 8°08'	2.10	209	- 29.8	86.0	Point in saddle N.W. of A.
	13	169°12'	- 2°42'	1.87	189	- 8.9	106.0	Break of ridge N.W. of A.
	14	179°16'	- 3°16'	0.67	68	- 3.9	111.9	Point in saddle N.W. of A.
	15	190°30'	- 2°50'	3.51	355	- 17.6	98.2	Head of broad draw N. of A.
	16	202°07'	- 1°12'	4.62	467	- 9.8	106.0	Point on slope N. side of draw.
	17	235°19'	- 2°36'	4.83	487	- 22.1	93.7	" " " " " "
	18	260°13'	- 2°43'	5.62	567	- 26.8	89.0	" " " " " "
	19	290°12'	- 3°08'	2.11	213	- 11.2	104.6	Cor. of fence at break of ridge.
	20	312°11'	- 4°18'	4.06	409	- 38.6	77.2	Point on fence on slope E. of A.
	21	348°19'	- 2°34'	1.62	164	- 7.4	109.4	Point at foot of steep slope S.E. of A.
	22	359°04'	+ 1°44'	0.16	16	+ 0.5	116.3	Summit of knoll S.E. of A.
	23	268°00'	- 6°20'	3.69	369	- 40.9	74.9	Pt. in bottom of draw N.E. of A.

Equipment:-

K+E Transit #12
Stadia Rod #6
Plumb-bob.
Reading Glass.

Dec. 11, 1909

Robson L. R
Clark, E.
Mason, J. Notes

NOTE:-

Azimuth of line I-IV = 138°08' From South; Length = 184.73 Feet. Auxilliary point, A, tied to this line by two distances and angle between lines A-IV and A-I.

Elevation of Sta. IV = 100.0 Feet

Used line A-I as line of zero azimuth in fieldwork. Azimuth of line IV-I for orientation of map.

$f = 101.0$ $(F+C) = 1.15$

Approximate sketch of survey on page 22. *

* This sketch should be similar to that for Prob. No. 8.

and record the magnetic bearing. Notes for recording should take the following form:

TAKEN IN FIELD							OFFICE COMPUTATIONS		
Inst. at		Sight	Azimuth	Mag. Bear.	Rod	Vert. Angle	H. D.	V. H.	Elev.
Sta.	H.I.								
A	4.5	B	123°16'	N 56°45' W	4.32	8°12'			
		F	331°50' *	S 28°10' ** E	6.45	4°08'			

(173) In traversing, the distances, horizontal angles, etc., should be observed both to the forward station and to the rear station. The readings at all stations throughout the traverse should be thus checked. Either the azimuth or back-azimuth of rear stations may be recorded. In the above notes the forward azimuth of A from F may readily be obtained by subtracting 180° from that recorded above (331°50' - 180°00' = 151°50'). So also, if the zero azimuth is taken as magnetic south, then magnetic bearing of AB checks within one minute with the azimuth, and the magnetic back-bearing of A_F likewise checks the azimuth of the line. These estimations should be made a part of the field work, checking observed values as the work proceeds. Computation of H.D.'s, V.D.'s and elevations are essentially office labors and are rarely carried out in the field.

(174) In taking measurements for topographical data the stadia possesses many advantages surpassing most other methods by reason of the rapidity and the lesser cost in time and labor required for a given amount of work. To get the topographical details over any

* Back azimuth (-azimuth + 180°)
** Back bearing (Forward bearing, FA, is N 28°10' W)

district by transit and tape or to get them where differences in elevation would be obtained by use of the engineer's lead, for example, it requires two parties, a transit party consisting of an instrument man and two chainmen who might also act as flagmen, and for the level party an instrument man and a rodman. These two parties would be obliged to work over the same territory, or one set of men working in two distinct organizations. But by stadia one instrument man and a single rod man (or two or more rod men can sometimes be employed to advantage) can obtain all necessary data in equal or shorter time.

For purposes of control either a triangulation system or a carefully determined traverse should first be laid out - this for horizontal control; while a series of bench marks should likewise be established, the same being connected with a suitable datum for a vertical control. The triangulation system or the traverse with the bench marks thus constitute a skeleton or frame-work to which the topographic details are related and to which they are joined as the work proceeds. The subject of Topographic Surveying will be treated in subsequent assignments - 32 and 33 of this course. But allusion is here made to the subject to set forth the possible advantage with which stadia methods may be applied in this sort of work.

The very skeleton in framework, be it triangulation or traverse, may be determined in the first place by stadia measurement. Such work, however, should be done with utmost care, double readings by stadia should be taken on all lines and observations

on bench marks, they should perhaps be done by means of the level on transit telescope, and made closely within the precision limits of such leveling.

References:

Tracy, pp. 308-310
Raymond, pp. 249,250.
Johnson, pp. 233-235
Breed & Hosmer, p. 168, Vol. II.

PROBLEM

Compute the H.D., V.H., and Elev. in the tabulated notes on page 6 of this assignment. $\frac{F}{i} = 100.2$; $F + c = 1.2$ ft.

UNIVERSITY OF CALIFORNIA EXTENSION DIVISION

Correspondence Course

Course 1B Elements of Surveying Swafford

Assignment 23

PROFILE LEVELING - CONTOURS - GRADE LINES

Foreword.

While the method of running profiles has already been given (see assignment 8), certain observations respecting profiles are made in the present assignment. We shall also treat here of contours and grade lines as related to leveling.

(175) When one is surveying for profile, it is of prime importance that one should select a well defined bench mark related to a datum of permanent character and connect these distinctly with the profile data.

Datum should be either mean sea level or some other reference plane wisely chosen. If the survey is near the sea-board or some government survey, a bench mark connected with the survey may readily be found. Other bench marks may also be established for purposes of convenience as the survey progresses by differential leveling. The bench marks should be as carefully selected as the datum plane. Their description and location should be well defined so that they may be readily found when required.

(176) The selection of suitable turning points is of prime consideration. These should be chosen because of their importance in carrying forward the line of levels run from bench mark to bench mark. Since the readings on turning points (and also on

bench marks) are always given to the one-hundredth of a foot and often to the thousandth of a foot, turning points must be selected with careful regard to stability, convenience, and, where they are also utilized as bench marks, with regard also to their permanency for such purpose. A turning point should be constructed which has the above named qualities and also which offers a projection of suitable shape so that the rod held upon it will in all positions be at the same elevation. For such a turning point, a spike or steel pin with a rounded head driven into firm earth, the top of a manhole cover, the higher edge of a stone coping, a door sill or curb stone, a railroad tie or rail may be used to advantage. In lieu of such artificial points many natural ones suggest themselves and may often be found in the route of a line of levels; for example, outcropping rocks, stones that are firmly imbedded, especially boulders, the projecting root of a tree, a stump, or a mark upon the trunk of a tree, etc.

If a turning point is desired as a bench mark, it is well to look to the matter of permanency, its description, and availability before adopting it for this capacity.

In any case where a turning point is to be established, care must be taken to read the rod to the proper degree of accuracy, since all subsequent readings of rod on the line and in fact the "Backsight" reading on the T. P. will affect the line readings from there on. Moreover, the "foresight" reading on a T. P. must be made before the level has been disturbed in its last setting and when the instrument is in station adjustment, the bubble

being precisely at center. Remember too, that the level set-up in accurate work is equidistant from two successive T. P.'s in order that any error in adjustment may be eliminated.

(177) You should choose points for stationing the instrument, where stations are not otherwise provided by the survey, so that the level may be used advantageously for taking the "backsight" and reading, if possible, so as to include several foresight readings. It will be seen then that a level station (location of instrument) will rarely or never be upon the line of levels, but rather at a convenient place to one side; if at a station higher than the turning point, not so far aside as to exceed the limit of "long rod"; if at a station lower than the T. P. certainly not so low that the line of sight cuts the ground below the foot of the rod. In the choosing of instrument stations the level man must be guided by experience and good judgment. A level plane may be swept out by the eye, the angle of slope may assist in judging. When the instrument is set up at any place, it should be approximately leveled according to a sight taken along the barrel of the telescope. Apart from these things, a man's experience determines the ease and rapidity with which he may work. Be alert and observant; get a low point on backsight reading where the grade is falling and a high backsight where the grade is rising - these enable a longer range of foresight in either case.

(178) Grade Line

The line which marks the slope of a road, railroad, or mine tunnel, ditch, etc. is called the grade-line. Such a line is

usually shown in its relation to a profile and is graphically represented on the mapped profile for convenience and study - and also for determining relations between the ground surface (profile) and the finished road-bed at any point along the line of road. Grade lines are sometimes expressed in degree of angle of slope (up or down) or by precent of grade. We say, for example, that the grade has a slope of 1/2° or 2°, 5°, etc., meaning that the slope is at the designated angle to the horizontal plane (the horizon). Commonly, we say that the grade is a 3%, a 5% or 0.2% grade. This last means that it rises (or falls) 3 feet in elevation at distance of 100 feet, 5 feet in elevation at 100 feet, 0.2 feet in 100 feet (for the above grades) and so on.

To represent graphically a grade line, a point is chosen, preferable at a convenient bench mark or at the station beginning any profile, and the elevation of the grade line is located then at 100 feet distant (by scale); the heights of the grade line is increased (or diminished) by the per cent amount; next a straight line is drawn through these two points. Should the grade change in degree (or percent), the point at which such a change takes place must be located upon the line (as graphed) and the change shown. The illustration, Figure 64, shows a profile with horizontal distances (stations) on a scale of 1 in. = 60 feet, and vertical heights (elevations) on a scale of 1 in. = 6 feet. The grade lines AB, BC, CD, indicate changes of grade along the route.

The height of the surface above grade line (or depth below) may be scaled off the profile chart at any point along the line.

Thus, at station 0 + 64, the height above grade is 2.3 ft., at 1 + 80 it is 3.1 ft., at 2 + 40 it is 4.2, etc.

This relation of grade line to profile will be further applied in crose-section work, Assignment 24.

(179) Contours

As differences in elevation along a line are graphically shown by a profile, so the differences of deviation over a ground surface are represented by contours.

A contour line, or simply a "contour", is the line showing the intersection of a horizontal plane with the ground surface; it may be defined also as a line of equal elevation over any area.

A clear conception of contours may be gained by conceiving an area consisting of a depression of considerable extent filled with water. Suppose now that the shore line of the water (as a lake) to be marked upon the map; this is the representation of the lake in plan upon a plane; the shore line is a contour at surface level of the land area surrounding the lake. Now imagine that the water in the lake subsides, say one foot; the new shore line will mark another contour upon the map, and so on; for each subsidence the shore line will determine another contour.

In case elevated portions of land are present within the limits of the subsiding lake, these elevated parts will appear as islands which rise higher and higher as the water falls; the contours of such high spots might be shown by ring-like lines encircling the rising land portions at various depths.

It will be seen that lines forming closed curves may represent the points of equal elevation in depressions and upon peaks. Again let us choose a given elevation in the sloping side of a

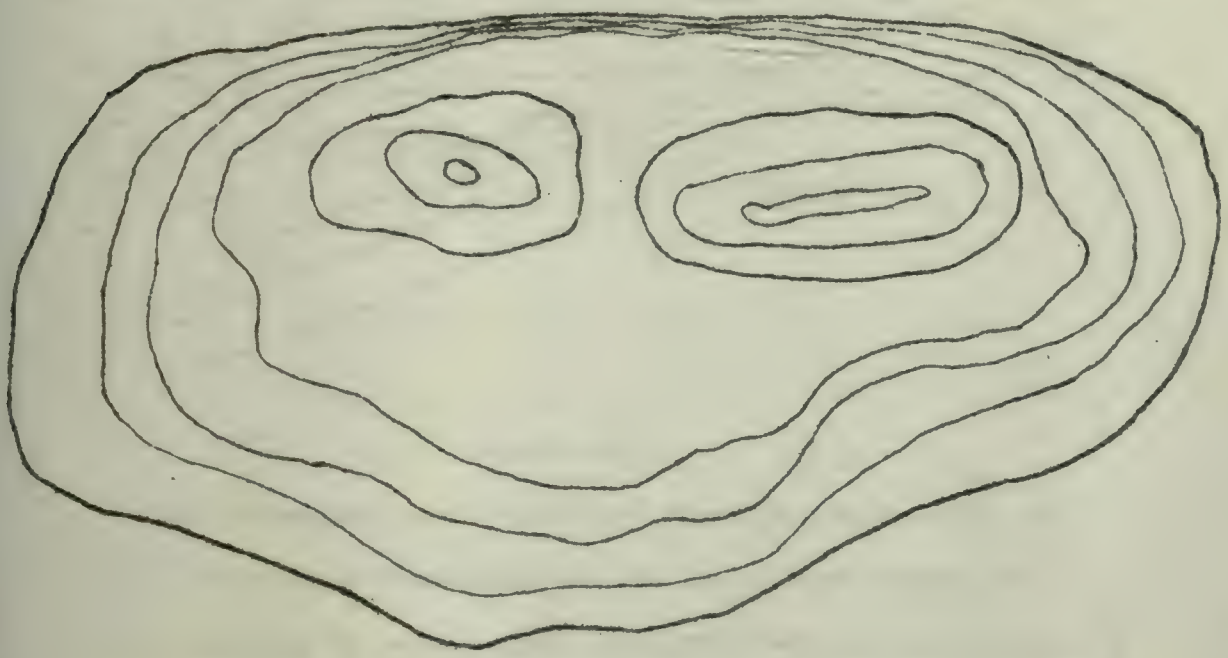


Figure 65

valley, at the lowest part of which we will suppose a stream flows. Keeping our feet on that elevation while we walk in the up-stream direction, we shall find that we approach nearer and nearer to the stream; at a certain point (where the elevation assumed equals that of the water in the stream, or of the banks on either side) we cross over to the opposite side. It must be kept in mind that we are following a path that is always at the same elevation above some assumed datum.

This line may be expressed grapnically by a contour line along a stream as shown in Fig. 66. The contours there represented show the 540, 545, and 550 foot contours along the stream as we go

up on one side, cross the stream at the given elevation, and return on the other side of the valley.

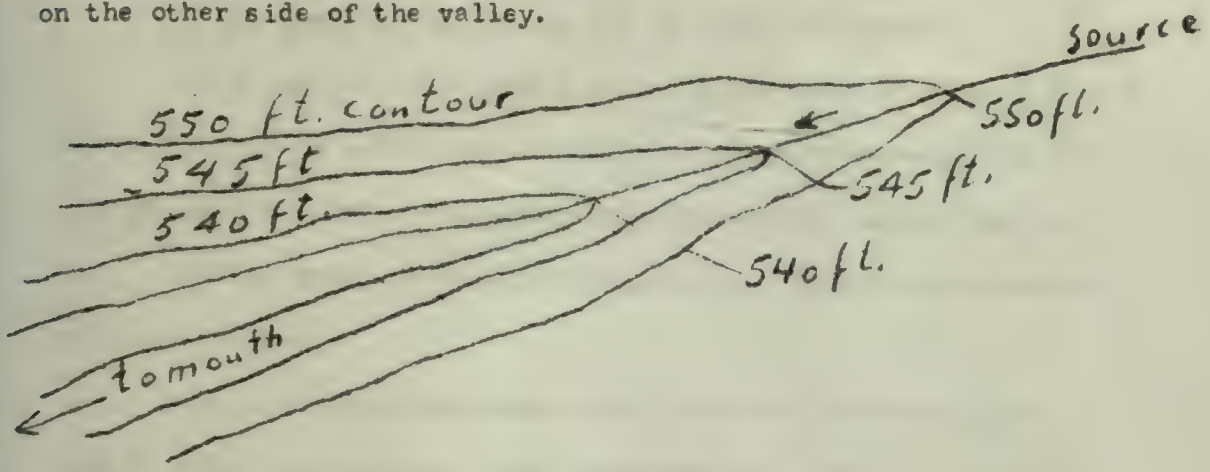


Figure 66

Over ridges or around hills the contours are shown as in Fig. 67. The dotted line indicates the back of the ridge; the contours supposed to be all at the same interval are more or less evenly spaced, at varying intervals; this shows that the hill

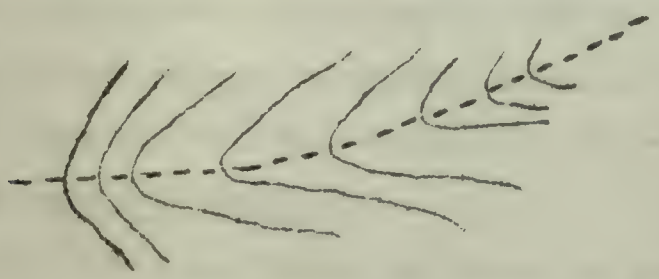


Figure 67

(as here represented) has different slopes at several points. The contours over hills are drawn at right angles to the ridges. It will now be well to call attention to the characteristics of contours in the following summary.

(180) Characteristics of Contours.

On any contour the points are at equal elevation.

Every contour will form a closed curve more or less irregular if the area is of sufficient extent. As a map is necessarily of limited extent, the contours may not close within such a limit.

About a peak, or within a depression, contours will appear as closed curves.

Contours cross each other only when they represent the relief of an over-hanging cliff, and then they show intersections.

Contours run up a valley on one side, cross a stream at right angles and return down the opposite side.

Contours around hills cross ridges at right angles.

On uniform slopes contours also present a uniform appearance as to direction and spacing. On steep slopes the contours are close together and on gentle slopes they are wide apart. On a plane or level tract, contours practically disappear, or at best are depicted, where possible, by parallel lines very widely spaced.

(181) Relation of Profile to Contours.

If a line - straight, broken, or curved - be protracted upon a contour map, it may be represented in relief as a profile. The elevations are in this case taken directly from the contour lines cut by the line of profile at the points of intersection. Thus profiles may be conveniently made in any direction upon the contour map; likewise lines may be protracted upon a contour map to show the course (direction and distance) of such lines at any

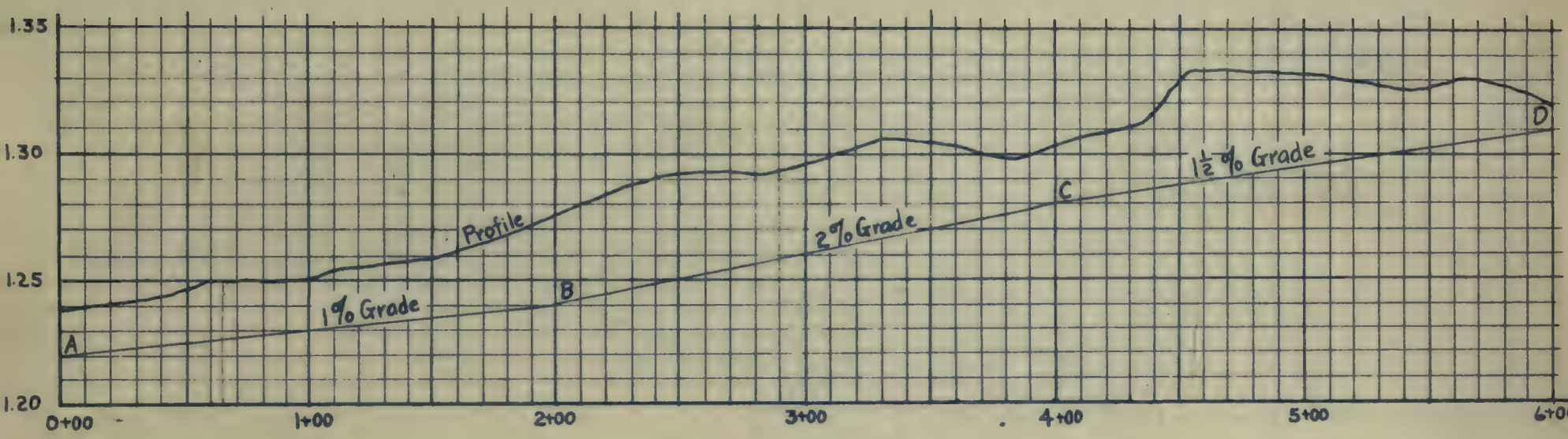
assumed grade, the grade being expressed in terms of elevation; this last may be reduced to per cent grade, where desired.

References:

Tracy	pp. 337-339
Raymond	pp. 245-247
Johnson	pp. 260
Breed & Hosmer	Vol. I.

Relation of Grade Line to Profile

Graphically Shown



Scales: { 1 inch = 60 feet Horizontal
1 inch = 6 feet Vertical

AGS

Figure 64.

PROBLEMS

1. Complete the following set of profile notes, draw the profile (sketching accurately) and show grade line of 2-1/2% grade.

Sta.	B.S.	H.I.	F. S.	Elev.	
B M_1	5.13			100.00	Note: In this set of levels B.M.$_1$, 100.00 ft. above datum has been previously determined; B.M.$_2$ was found by differential leveling; the stations are taken in intervals of 50 feet.
0			6.3		
1			7.1		
2			8.8		
T 3 P_1	2.16		9.80		
4			5.9		
5			7.9		
6			10.2		
T 7 P_2	1.26		12.02		
8			4.2		
9			5.9		
10			6.9		
T 11 P_3	3.04		11.75		
12			4.7		
13			6.4		
14			5.3		
B 15 M_2			7.23		

2. Scale off the ground surface elevations above grade from chart, Fig. 64 at 2 + 90, 3 + 30, 3 + 80, 4 + 35, 4 + 55, 5 + 40.

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UNIVERSITY OF CALIFORNIA EXTENSION DIVISION

Correspondence Course

Course 1B Elements of Surveying Swafford

Assignment 24

CROSS-SECTION LEVELING, VOLUMES, EXCAVATIONS

Foreword:

The aim of this assignment is to give a general treatment of the use of level readings over an area for obtaining depths of cut or height of fill, these in turn to be used to measure quantities of earth to be moved in either case. A review of the methods of computing volume will also be made with special application to such work as is likely to confront the engineer.

(182) Cross Section

A cross-section, as used in the sense here considered, is found by determining the area of a transverse section of road, railroad, canal, etc., at right angles to the line of such road, etc This area is bounded by the bed of the road, the angle and extent of the sloping sides, and the profile of section at the ground surface.

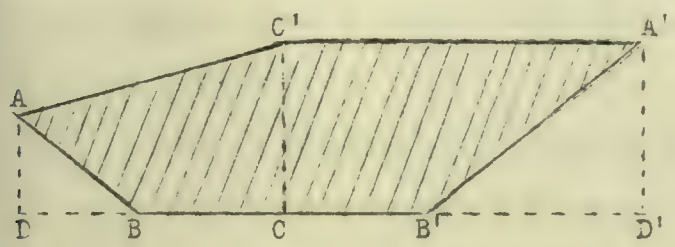


Fig. 68a

In Fig. 68a is shown the cross-section of a roadway. B B' is the width of road at the base; A B and A' B' are the sloping sides of an excavation (called a cut); C C' is the height of the ground above the center-

line of the road when finished; AD is the elevation of the ground above the road-bed where it cuts the surface on the left; and A' D' is the corresponding measurement on the right. The bounding lines of the section are then AC'A'B'BA. Had the section been an embankment instead of a cut (i.e. had the road been built up on the surface of the ground instead of being dug away) the appearance might be similar to that shown in Fig. 68b. Here the road-bed is at top of embankment. In Fig. 68a it is at bottom of excavation. The areas, shown by the shaded portion, in either case are similarly bounded, AC'A' being the profile of the section at the ground surface.

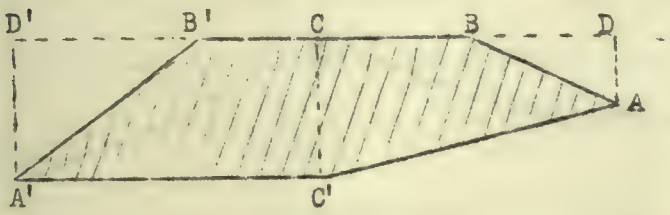


Fig. 68b

For the purpose of computation, it is necessary to know the following: Width of road-bed BB', elevation of CC'(depth below center or height above center), vertical heights AD and A'D', and distances out from center CD and C D'. The area can then be found by computing the area of each triangle composing the figure and taking the sum, thus:

Area of Triangle ABC = 1/2 BC x AD

" " " A'B'C = 1/2 B'C x A'D'

" " " ACC' = 1/2 CC' x CD

" " " A'C'C = 1/2 CC' x CD'

The sum of these partial areas is:

$$\frac{BC \times AD}{2} + \frac{B'C \times A'D'}{2} + \frac{CC' \times CD}{2} + \frac{CC' \times CD'}{2}$$

Observe that BC = B'C. each being half the width of road-bed, and CC' = CC'. Therefore, the above summation may be written in simpler form:

$$\frac{(AD + A'D')\ BC + (CD + CD')\ CC'}{2}$$

Note:- The distances CD and CD' are really measured at the surface of the ground in any case, the points C, D, and D' being beneath the ground and inaccessible for this purpose.

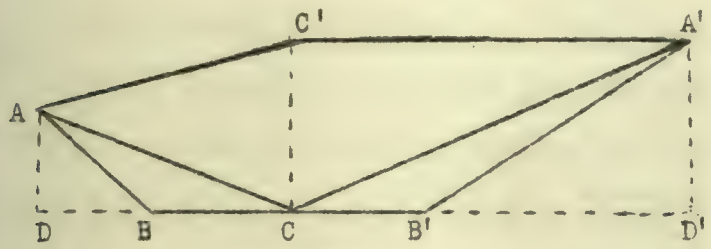


Fig. 68c

We have given this geometric solution of the problem at the beginning, in order that you may fully understand its nature and the data required in its determination. The work of the level party in cross-sectioning is given as follows:

The level is set up at some suitable place where readings may be taken upon a rod held at C', at A, and at A' for any section. Preferably, similar points on two or more other sections may also be taken. This setting must be convenient to some bench mark connecting with the general profile levels previously determined along the line of road. The H.I. is obtained by adding the B.S. reading on B.M. to the elevation of B.M. The difference between

the elevation of the road-bed and the H.I. is the grade-rod (the rod-reading for grade). The rod is now held upon C, and this reading subtracted from grade-rod gives the cut or fill at the center.

Next, determine the elevation above grade of A and A' (AD and A'D' in the figure). To do this the rod must be held at a point in a line at right-angles to the line of road from C; the distance out is one-half the width of road-bed plus the height multiplied by the slope. Inasmuch as the distance is dependent upon the height the correct position can only be determined by trial. Hence, we estimate the point where the sloping side of the road would cut the ground surface. A rod-reading taken here, subtracted from grade-rod, will give the elevation above road grade. Compute the value of CD from the formula:
$CD = 1/2\ w + hs$, where w = width of roadway, h = height, and s = slope. Should this value agree with the measured distance, the rod has been held at the right place. Otherwise the rod must be moved toward or away from C until the desired agreement between computed and measured distance is found. A stake driven at the correct point as obtained above is called a slope-stake. On this stake is marked the amount of cut or fill (AD, A'D') at that point. Since this can only be done by trial, much patience and careful judgment is the price of success. By much practice, one may become adept at setting slope stakes, but it is well to understand fully the theory and purpose in this work.

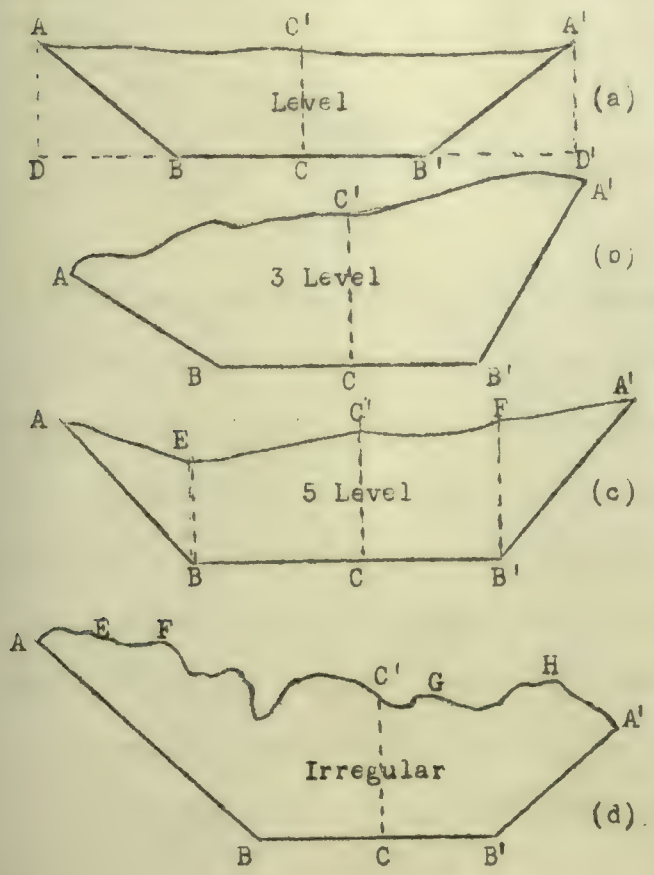
183) Slope

This is usually expressed in a ratio of the distance horizontal

to the distance vertical; as 1 to 1 (one foot out to one foot up or down); $1\frac{1}{2}$ to 1 (a common slope for compacted earth); and 2 to 1 (loose earth). A slope of 1 to 1 or steeper, even to vertical sides is a slope for rock sides, loose to solid.

(184) Sections

Sections are of four different kinds, according to the nature of the profile. These are: (1) level section where the section profile is horizontal; (2) three level section where the profile has two different slopes from the center stake; (3) five level section in which case the rod must be read at five points along the section profile; (4) irregular section where the nature of the profile requires (for accuracy) several rod-readings. These are illustrated in Fig. 69. Of these the level section is the simplest.



The method of computing the area of the three level section has already been explained. To compute the area of five level section in general (referring to the Fig. 69, c) divide the section into triangles, AEB and FA'B', and trapezoids, EC'CB and C'FB'C, the data

for which are readily determined. For irregular sections, other methods are usually employed, the common way being to plot the section to a scale of convenient size and measure the area with a planimeter. The three level and five level sections may also be measured by planimeter applied to a plotted figure, but a level section is always most easily computed by dividing into the two simple trapezoids, AC'CB and C'A'B'C, as shown in Fig. 69 (a). The illustrations represent cuts and by inverting the figures they will represent embankments as well.

(185) A side-hill section is one where the slope of the ground surface intersects the road-bed. Such a section requires both cut and fill at that point of the road. The shaded portions in Fig. 70 show the cut (to left) and the fill (to right). This is generally considered an economical mode of construction, as the material removed from the part ABC may be used in filling the part CB'A'; hence roads, railways, and ditches are conveniently carried along hill-sides. Much haulage is saved in such construction. Cross-sections are taken at intervals of 100 feet, or 50 feet, or any other convenient interval depending upon the character of the ground surface, nature of material to be moved, and other varying considerations. Suppose that the interval between any two adjacent sections is 100 feet, then the material embraced by the two sections (in this case called end-sections), the ground surface, the road-bed, and the

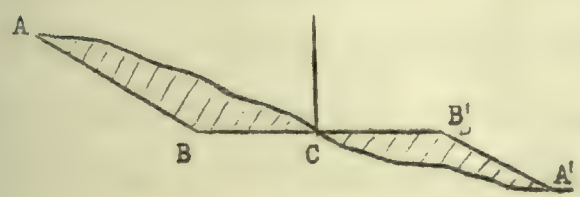


Fig. 70

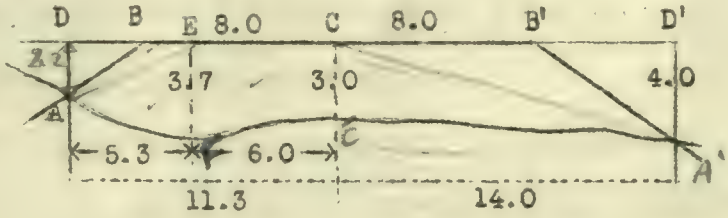
sloping sides forms a prism. The bases of the prism are the end-sections, the mean of which may be regarded as the mid-section (or right section) of the prism. The altitude of the prism would be 100 feet in this case.

By applying the geometric formula for the volume of the prism, $V = L \frac{A + A'}{2}$ the amount of the cut or fill may be computed.

(186) Example

Area of end sections, A = 225 sq. ft., A' = 235 sq. ft., L = 100 ft., then $V = \frac{100 \times 225 + 235}{2} = 23000$ cu. ft., and dividing by 27 (no. cu. ft. in 1 cu. yd.), we have 852 cu. yards, which is the amount of earth to be moved. Specimen notes for cross-section work are shown on Sample Notes, plate 23, for a three level section. You will note that the stations (first column) start at the bottom of the page and run upward. A full station is 100 ft., so that 37 + 00 indicates a full station; next is 33 + 00, then 38 + 60, 39 + 00, etc. The second column gives elevation at C, the third gives the grade elevation at C. On the opposite side of the page the difference of these two elevations gives the depth of fill at C; at left the upper number gives the height of A above grade, the lower number the distance out from C; the letter f indicates fill. At the right, we have the same notation repeated for A'. Also an additional reading is shown at the left 6.0 ft. out from C with difference between reading at this point and C of 3.7 ft., an irregularity that sometimes occurs.

We will now plot this section (Sta. 37+00), and proceed to the computation of the area for the purpose of further illustration.



Trapezoid ECC'F: $6 \times \frac{3.7 + 3.0}{2} = 20.1$ sq. ft.

Triangle BEA: $\frac{(8.0-6.0) \times 2.2}{2} = 2.2$ " "

Triangle AEF: $\frac{3.7 \times 5.3}{2} = 9.8$ " "

Triangle CB'A': $\frac{8.0 \times 4.0}{2} = 16.0$ " "

Triangle CC'A': $\frac{3.0 \times 14.0}{2} = 21.0$ " "

Total area of section = 69.1 sq. ft.

For section at station 38 + 00 the computation is as follows (abbreviated form):

$$\frac{8 \times (1.2 + 3.0) + 2.3 \times (9.8 + 12.5)}{2} = 42.5 \text{ sq. ft.}$$

To compute the volume of the embankment between end-sections at station 37 and station 38, we first find the geometric mean of the two sections. This is:

Mean section = $\sqrt{69.1 \times 42.5} = 54.2$ sq. ft. Then the volume on 100 feet of line is: Vol = $l\sqrt{A \times A'} = 100 \times 54.2 = 54200$ cu. ft., or 200.4 cu. yds.

187) Excavations

Cross-sectioning is also employed in determining the amount

of material removed from excavations or the amount of material required to fill depressions over a limited area, as in the case of a pit (especially a borrow pit, a cellar, etc.)

In this work the surface area is divided into squares or rectangles (sometimes into triangles) of convenient size. Suppose the case of a borrow pit, Fig. 71, divided into squares 10 ft. x 10 ft., a few portions being shown as triangles for purposes of illustration. The dividing lines are numbered to right in the figure, 0, 1, 2, 3, etc., and downward, A, B, C, D, etc., so that references may be readily made to the points of subdivision of the area (ground surface); as the upper left hand corner is A0, B4, D2, G6, F3, and so on.

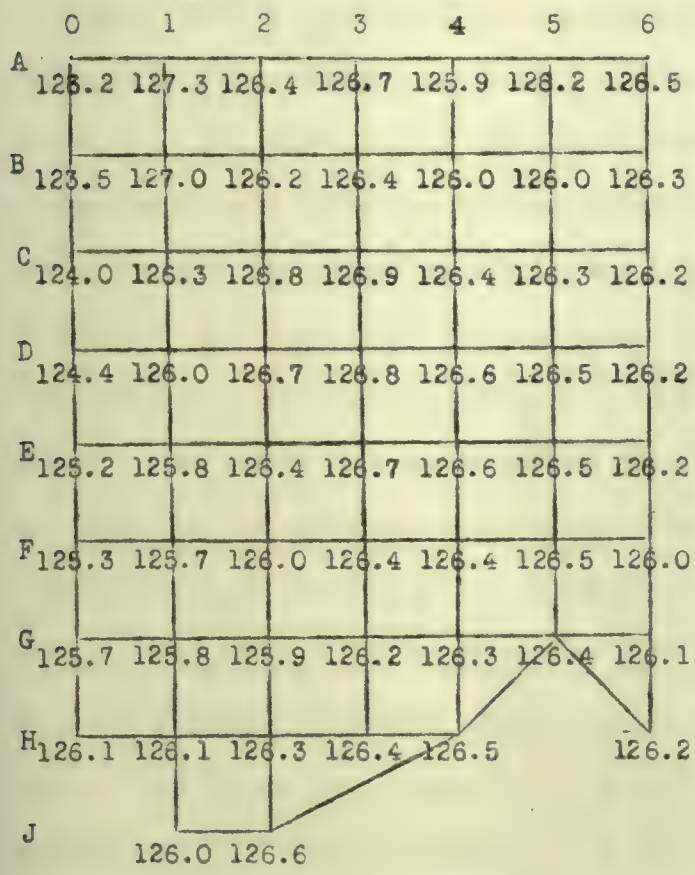


Fig. 71

The divisions are staked off on the ground and the designation as above marked upon a stake driven at each point as also its elevation above some convenient B.M. to which all rod-readings are referred.

The rod-readings are taken at each point and recorded in the notes of the survey, and may also be shown upon a plot of the area and the depth of cut (or fill) marked upon the stake at that point. The rod is always held upon the ground, not on the stake. A level instrument, or a transit with a level on the telescope, is set up at a convenient place where observations may be made upon a rod held upon the B.M. and upon the cross-section points and rod-readings taken as follows: Back-sight on B.M., fore-sights on A0, A1, A2, and so on. The elevation of the finished bottom of the pit (or embankment) above datum having been established, and the H.I. of the instrument determined by adding the B.S. to the B.M., the rod-reading, or H.I. for grade is computed. Then the F.S. readings taken at the several points subtracted from the H.I. for grade will give the depth of cut, or height of fill at each point. We now have the elevation of these points above the bottom of the pit, or the height of the top of the finished embankment measured from the ground surface. You will observe that we now have the dimensions of a number of prisms, each right section of which is the same in the length and breadth. The dividing up of the ground surface in uniform figures both simplifies the measurement, and as you will presently see, lessens the computations as well. The length of each edge of any particular prism is given by the elevation of its corners, obtained as explained above. But we require the mean length of each prism which multiplied by the area of its right-section will give the volume of the prism. To obtain the

mean length of the prism add together the elevation of the several points of the ground surface and divide the sum by the number of points, i.e. the number of edges - usually four.

Referring to Fig. 71, the prism AO, A1, B1, BO, would be found by taking 1/4 the sum of elevations at AO, A1, BO, B1, and multiplying by 100 (= 10' x 10', the dimensions of the right-section); the prism A1, A2, B2, B1 likewise has a volume equal to $\frac{100 \times (A1 + A2 + B1 + B2)}{4}$, and so on. You will see that the right section is uniform over the whole area covered by squares. The right-section H2, H4, J2 is readily computed, being a triangle $\frac{10 \times 20}{2}$, hence 100 sq. ft.; while the triangles G4, G5, H4, and G5, G6, H5 have each a right section of 1/2 (10 x 10) = 50 sq. ft.

A further study of Fig. 71 will make plain that if we take the elevations of all cross-section points, we will have used AO once, A1 twice, B1 four times; or, from another point of view, at every exterior corner the elevation is used once; every outer point other than these is used twice; while every interior point is taken as many times as there are squares having a common corner.

The depth of cut or fill for each point may be listed in columns which are headed 1, 2, 3, 4, to show the number of times each quantity enters into the computations. We shall list them by the coordinate numeration, but in actual work the depth of cut (or fill) should take the place of this designation. This is given here, not as a numerical example, but as a general illustration of the method.

The total in column 1 is taken 1/4 times; in column 2, 2/4 times; in column 3, 3/4 times; in column 4, 4/4 times. The grand total of these is multiplied by the area of cross-section of one prism (10' x 10' = 100 sq. ft.); this, divided by 27, gives the number of cubic yards. To this must be added the irregular portions, consisting of the three triangular sections partially computed.

1	2	3	4
A0	A1	G4	B1
A6	A2	H2	B2
G6	A3	H1	B3
H4	A4		B4
J2	A5		B5
J1	B6		C1
H0	C6		C2
	D6		C3
	E6		C4
	F6		C5
	G5		D1
	H3		D2
	G0		D3
	F0		D4
	E0		D5
	D0		E1
	C0		E2
	B0		E3
			E4
			E5
			F1
			F2
			F3
			F4
			F5
			G1
			G2
			G3

Irregular Sections:

(a) H2, H4, J2

$$\frac{10 \times 20}{2} = 100 \text{ sq. ft.}$$

16.3
16.5
16.6

3)49.4 = 16.5

$$\frac{16.5 \times 100}{27} = \text{..... cu. yd.}$$

(b) H4, G4, G5 and G5, G6, H6 (c)

16.3
16.4
16.4
16.1
16.2
16.5

6)97.9 = 16.6

$$\frac{16.6 \times 100}{27} = \text{..... cu. yd.}$$

PROBLEMS

1. From ~~data given in notes, Plate 23,~~ the following data compute the area of sections at stations 40 and 41 and also the volume in cu. yds. of earth to be moved.

Station	Surface	Grade	Left	Center	Right
41	118.0	110.9	C 8.0 / 22.0	7.1 c f	C 6.2 / 19.3
40	114.7	110.1	C 6.0 / 19.0	4.6 -	C 1.2 / 11.8

Width of roadway in cut 20 feet.

2. From data given in the illustration, Fig. 71, compute the amount of material to be excavated to bring the pit to a depth having a uniform elevation at the bottom of the pit which will be 110 feet above datum. The cross-section checks are 10' x 10'. Follow the order given in the columns marked 1, 2, 3, 4 on page 12 of this assignment.

References:

Tracy	pp.428 - 434
Raymond	pp.275 - 280
Johnson	pp. 394 - 399
Breed & Hosmer	pp. 240 - 244

UNIVERSITY OF CALIFORNIA EXTENSION DIVISION

Correspondence Course

Course 1B Elements of Surveying Swafford

Assignment 25

VOLUMES of CUTS and FILLS (Continued)

Foreword

Extended consideration of cuts and fills and additional methods of computing volumes will be treated in this assignment, as well as the estimation of quantities in grading from contours.

(188) Over extended areas the methods of computing volumes of earthwork to be moved are modified to suit the greater magnitude of the operation. This is done especially where an approximate value suffices in the estimate. But in some instances the more exact method by the prismoidal formula is both expeditious and readily applicable, and gives more accurate results. The end-area method is sufficient for most estimates and gives results often within the desired limits of accuracy. The volume of a prismoid is $V = \frac{L}{6}(A_1 + 4A_m + A_2)$. If the dimensions for these computations are taken in feet, the volume will be in cubic feet, which may then be reduced to cubic yards (the common unit for expressing volumes of earthwork) by dividing by 27. In the formula L is the length of the prismoid; A_1 and A_2 are the end sections; A_m is the mid-section. This mid-section is not the mean of the end sections, but is found by taking the mean values of dimensions of the two end sections or by actually taking the necessary data at mid-position for determining the section area at that part.

PROB. NO. 19. CROSS SECTIONS FOR ROADWAY.

(23)

Station	Center Elevation Surface	Grade	Remarks
			Equipment. Jan. 10, 1910.
			Transit #61 Cool + clear.
			100-ft. steel tape Jason, C. L.
			50-ft. mat. tape. Clark, M. Y.
			Chaining pins Harris, K. Ch.
			Hammer
			Stakes
			Level Rod #18
			Roadway 16 ft. + 20 ft.
			Side slopes 1½ : 1.
41+00	118.0	110.90	In this column should
40+00	114.7	110.10	be entered the classifi-
+85	112.3	109.98	cation of material
+10	109.4	109.38	to be excavated, etc.
39+00	109.0	109.30	
+60	108.1	108.90	
38+00	106.0	108.30	
37+00	104.3	107.30	

Left		Center	Right
C 8.0 / 22.0		7.1	C 6.2 / 19.3
C 6.0 / 19.0		4.6	C 1.2 / 11.8
C 4.0 / 16.0		2.3	C 0.0 / 10.0
C 2.0 / 13.0		0.0	F 1.6 / 10.4
C 1.2 / 11.8	0.0 / 4.5	0.3	F 2.2 / 11.3
F 0.0 / 8.0		0.8	F 2.6 / 11.9
F 1.2 / 9.6		2.3	F 3.0 / 12.5
F 2.2 / 11.3	F 3.7 / 6.0	C 0 3.0	F 4.0 / 14.0

The cross-sections are usually taken at intervals of 100 feet, 50 feet, or 25 feet. Where the prismoids thus determined are fairly uniform, it is practicable to unite two consecutive stations, thus making the cross-sectional area of the intermediate station the mid-section of the combined prismoid thus formed; in this case the value of L will be 200 feet for full stations, or 100 or 50 for 50's and 25's.

By referring to Fig. 72, it may readily be understood that the corresponding dimensions of the mid-section may be obtained by taking the mean of the end-sections.

$C_m = \frac{C_1 + C_2}{2}$, $d_m = \frac{d_1 + d_2}{2}$, $h_m = \frac{h_1 + h_2}{2}$, etc. The width, w, of the roadbed is the same throughout, hence no change is made for this dimension at the mid-section.

In computing the cut or fill over large areas the work of computing is greatly simplified by checking off the area graded in dimensions that lend themselves to ready reduction.

The general formula for volume in excavations or fill is

$$V = \frac{A}{27} \left(\frac{\Sigma h}{4}\right)$$

in which A = ground area of a uniform section, Σh = the summation of depths of cuts (or heights of fills). Dimensions are taken in feet, and reduced to cubic yards by dividing by 27. Now write the formula in the form

$$V = \frac{A}{27 \times 4} (\Sigma h)$$

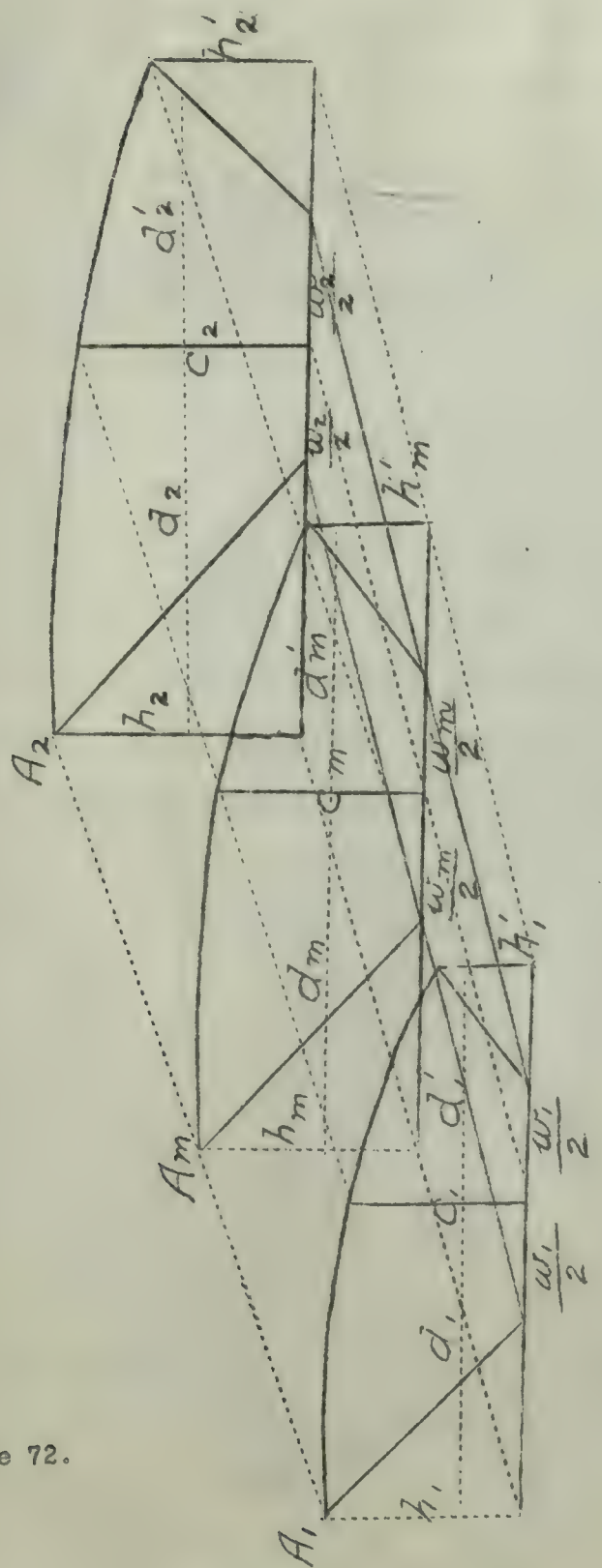


Figure 72.

If the checking of the surface be in rectangles 30' x 36' and h be taken to the nearest tenth of a foot, it may be seen by substituting above values that

$$V = \frac{30 \times 36}{27 \times 4} (\Sigma h) \text{ cu. yd.}$$

$$= \frac{1080}{108} \quad (\Sigma h) \text{ cu. yd.}$$

$$= 10 \Sigma h$$

Which means that the volume is found simply by summing the h's and dropping the decimal point. The surface might also be checked 18' x 60', or 12' x 90'. Or, by retaining the decimal point, the dimensions of the rectangular checks may be 9' x 12', or 6' x 18'.

By giving each problem a little thought before beginning the collecting of data it is often possible to simplify the work.

(189) Still another method of finding the volume of earthwork in grading is found in the use of contours as laid down upon a topographic map of the area. This method is especially applicable to

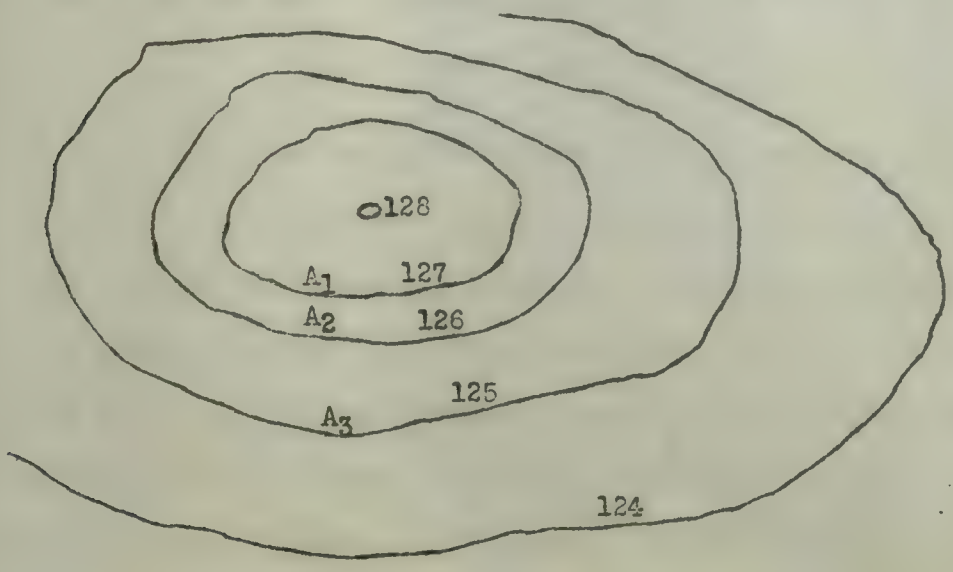


Figure 73

large areas, or for estimates generally. Take, for instance, the case of a hill that is desired to be reduced to the level of the surrounding terrain. In Fig. 73 is shown a hill depicted by contour lines having an interval of 1 foot. It is desired to reduce this to a level of 125 foot elevation. The area of each plane of horizontal section would best be determined by means of a planimeter. This is an area measuring device which will be discussed fully in a later assignment (Assignment 30). That part above contour 127 to the crown of the hill is regarded as a cone, the base being the area of the section at contour 127. The volume of this part will be equal to 1/3 h A_1 ($= \frac{h}{6}(2)A_1$). To find the volume of that part embraced between contours 127 and 125 it is necessary to know the areas A_1, A_2 and A_3. Here it is better to use the prismoidal formula regarding area at section of contour 126 as the mid-section. The volume of this portion will be then $V = \frac{h}{6}(A_1 + 4A_2 + A_3)$; hence the total volume is expressed as follows:

$$\text{Total Vol.} = \frac{h}{6}(3A_1 + 4A_2 + A_3).$$

It is sometimes desirable to grade a large surface, changing its slopes and elevations for landscape purposes. For this work the contours are determined and sketched upon a map of the area with a suitable interval. The contours of the finished surface are then drawn upon the map, preferable in a color different from that of the primary contours. The drawing of the two contour systems over the same area shows prismoids whose lengths are the contour interval and whose end areas may readily be measured by

planimeter. (Assignment 30) If it is desired to apply the prismoidal formula, two adjacent prismoids may be combined and the common section will constitute the mid-section. In Fig. 74 the heavy lines depict the contours with a 5 foot interval as drawn

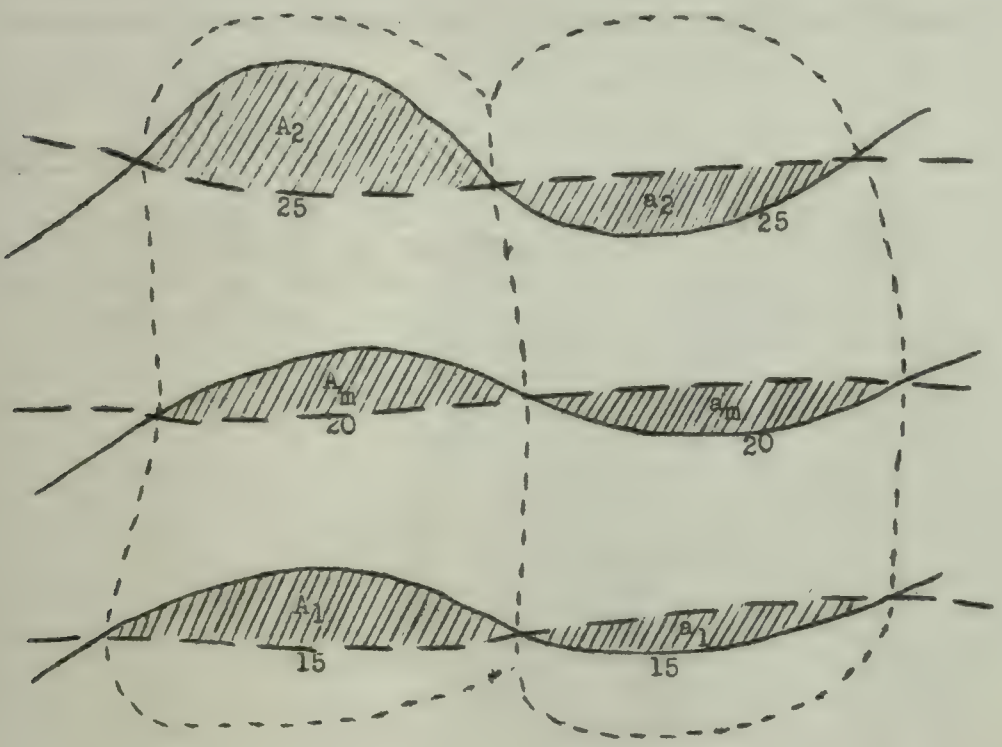
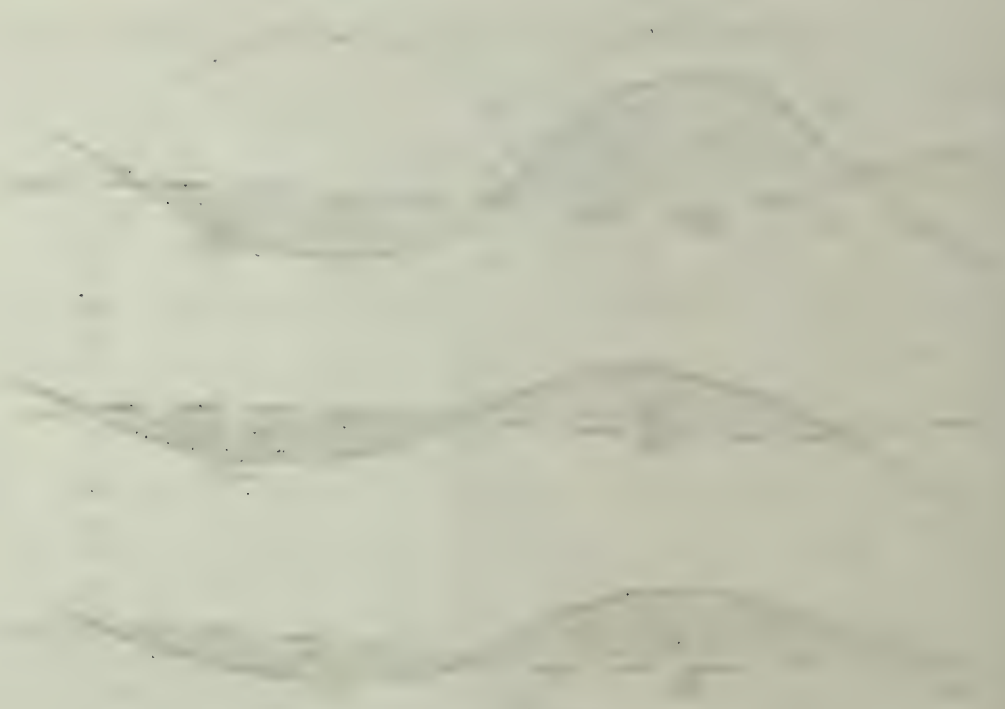


Figure 74

upon a map of an area to be reduced to a grade as shown by contours represented by the long-dashed lines. The sections in each prismoid are shaded, the section-areas may be found by planimeter or otherwise. The volumes will be V (cut) = $\frac{10}{6}(A_1 + 4A_m + A_2)$,

$$V(\text{fill}) = \frac{10}{6}(a_1 + 4a_m + a_2)$$

Here the material removed by cut is conveniently used to make the necessary fill, any excess of course to be removed



Other applications of these methods may be employed on roads, dams, embankments, etc. The ingenuity of the engineer may suggest a large variety of uses of such methods, by which much time and money may be saved; especially is this true in computing earthwork in estimates where a high degree of accuracy would prove expensive.

References:

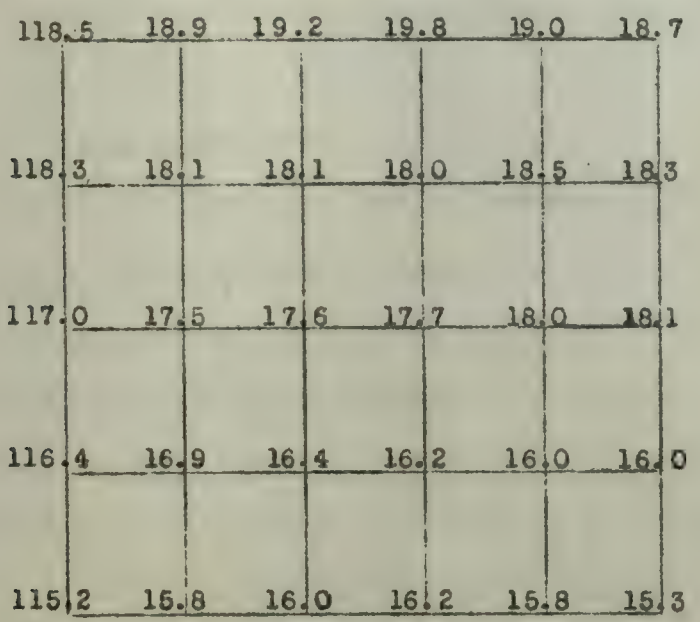
Raymond, pp. 281-286
Breed & Hosmer, pp. 390-395
Johnson, pp. 399-405
Tracy, pp. 435

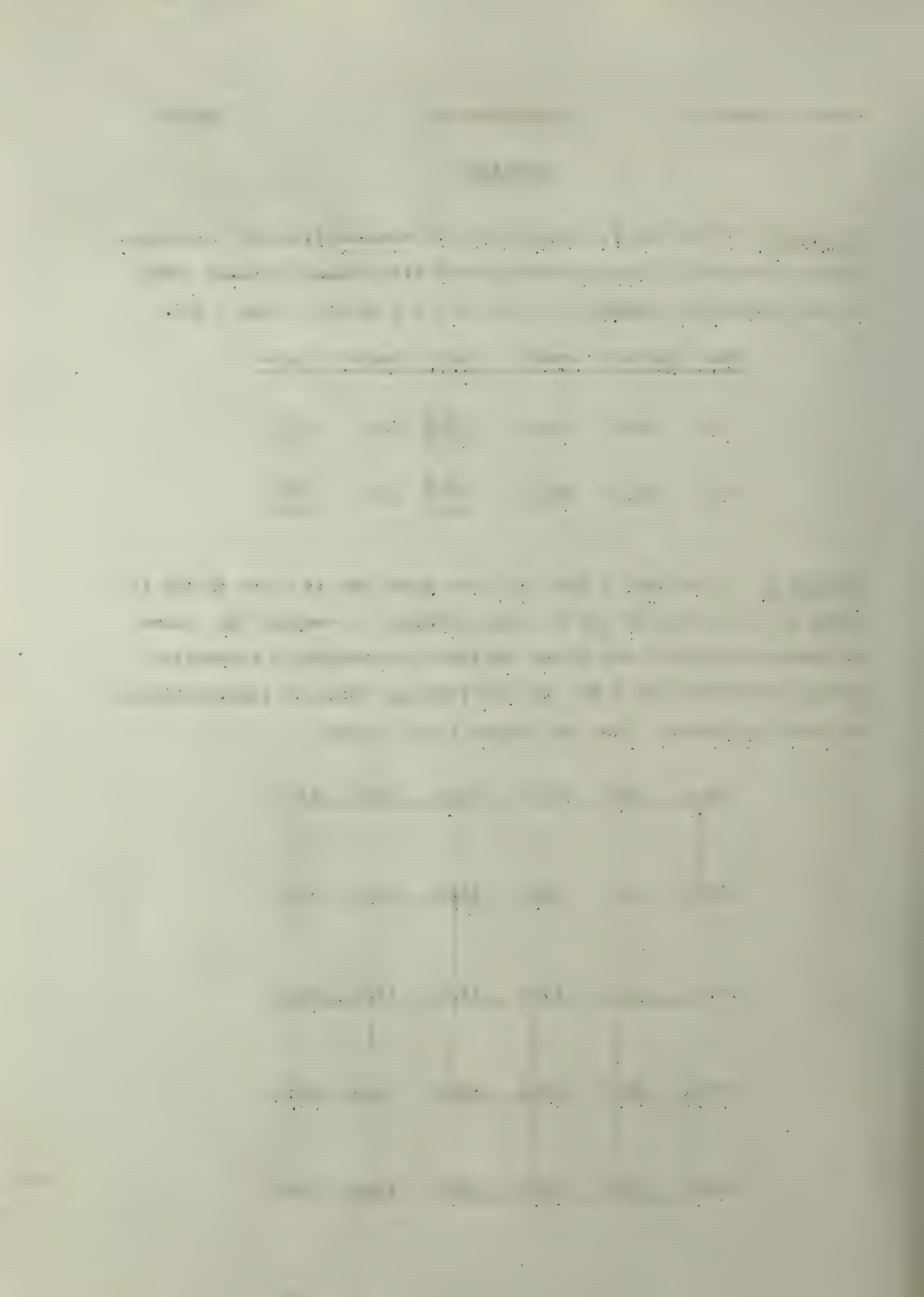
PROBLEMS

Problem 1. Given the following data of cross-sections of a roadway. Compute the area of the mid-section and find volume in cubic yards by the prismoidal formula. L = 100 ft., w = 20 ft., slope = 1/1.

Sta.	Surface	Grade	Left	Center c	Center f	Right
72	33.9	26.5	8.5 / 18.5	7.4		4.8 / 14.8
71	30.2	25.0	6.3 / 16.3	5.2		2.6 / 12.6

Problem 2. A lot 150' x 144' is below grade and is to be filled to bring it to a level of 125 ft. above datum. To compute the volume of material required the ground surface was checked in rectangles having dimensions 30' x 36' and rod-readings taken at intersections, as shown on sketch. Find the volume in cu. yards.





UNIVERSITY OF CALIFORNIA EXTENSION DIVISION

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Course 1B Elements of Surveying Swafford

Assignment 26

CITY SURVEYING

Foreword

A general consideration of City Surveying and the laying out of streets and city blocks with their grades will be considered in this Assignment.

(190) Instruments

In general the instruments used in city surveying should be of a higher degree of refinement than those used in country, farm, and highway work. This is partly for the reason that the property - lands, buildings, and other municipal improvements - is of greater value, and partly because of the permanent nature of all structures of a public character. On the farm, the value of the land covered by survey is usually a few hundred dollars per acre, while city lands reach a value in many cases of hundreds or even thousands of dollars per foot of frontage upon the street. Furthermore, many buildings are built upon the street line and upon the property lines that divide one landowner's holdings from those of his neighbor. Besides, the grades of streets, surface and underground drainage systems, sewers, waterpipes, curbs, sidewalks, andthe like, must be determined with great accuracy. The quantities obtained in the survey enter into computation in estimates of costs of materials and construction that run into large and often enormous sums.

Hence, in country surveys, the compass and chain usually suffice, and at best a good transit and a tape. In city work, however, a more refined transit, reading to 30, 20, or 10 seconds, and a tape carefully compared with a standard is necessary. Measurements to a high degree of accuracy should be taken with both instruments. With the transit angular values to the nearest 5 or 10 seconds should be obtained. With the tape measurements to the nearest thousandth of a foot are necessary.

On much city work leveling should be made to the nearest hundredth of a foot, and on bench marks to the nearest thousandth. For the latter a precise level and a straight, carefully graduated rod, having a vernier reading to thousandths, should be used. You should consult the descriptive catalogues of instrument makers showing the high-grade instruments made by them. All makers turn out precise instruments possessing features of special excellence, and intelligent selection cannot be made without a knowledge of several different makes.

(191) Most city surveying is conducted in cities already built up, where a few hundred or a few thousand people have come to live together before the necessity for accurate surveys has been realized. In some cases, however, a city may be laid out at the start, and much of the important basis lines, angles, and grades established before any population has come upon the land. In the former condition there are many things that may interfere with a desired or proper basic survey; in the latter, these interferences being wanting, the character of the beginnings in a city survey often

devolves upon the engineer whose duty it is to devise and perfect a suitable plan.

(192) In planning a city the size of the blocks is usually the first consideration, as many things connected with convenience in the residence districts, such as lawns, garden, out-buildings, etc., appeal to the early settlers in a new town, while many considerations affect the business district. But by far the most important matters for careful consideration are the widths and grades of streets, surface drainage, and sewers and sewage disposal. The directions of streets and their relation to the cardinal points of the compass also deserve attention.

Streets in the business district should be generally 80 to 120 feet in width with small blocks of about 225 feet square divided by an alley parallel to the main streets of 15 feet width. In the residence district blocks may be of 300 to 400 feet in length, the alley may be dispensed with, and streets narrowed to 60 feet. The subdivision into lots may be made 25 feet wide in the business district and 40 to 50 feet wide in the residence section.

(193) The position of a town site is usually determined by one or more of several considerations. These are: nearness to a body of water or navigable stream or to railroad station, or, in some cases, to a political division. Or the site may conform to a subdivision of a U. S. Land Survey. To such original, determined location there may, of course, be added from time to time other territory that may render the original plan more or less irregular

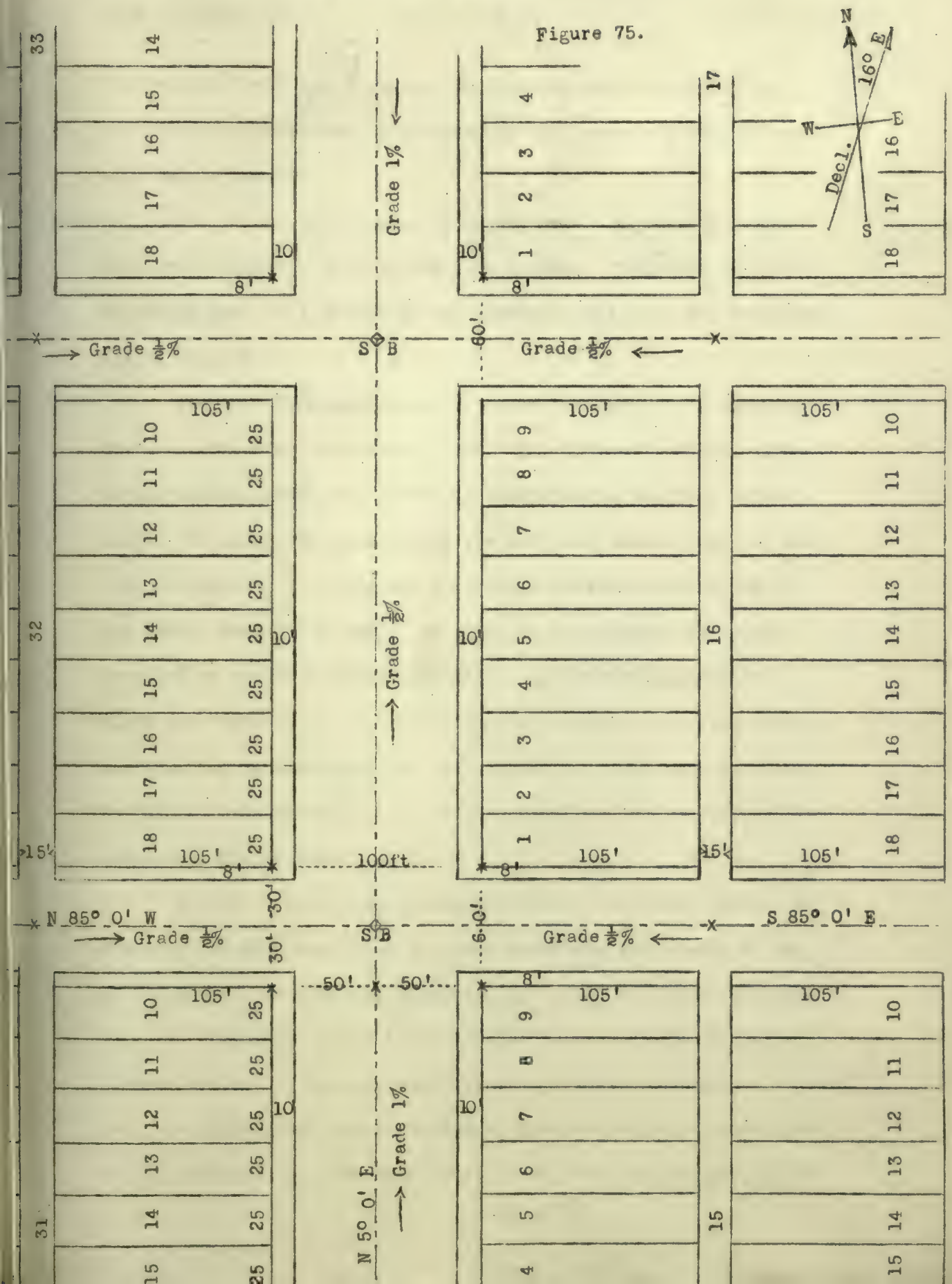
and introduce problems that call for the best skill and judgment of the surveyor. Such problems relate to the streets, grades, surface and underground drainage, sewers and sewage disposal, as was the case with the original site.

The matter of defining these things is largely in the hands of the engineer employed upon the surveys, but the final disposal of these matters is through act of the governing body of the town or city and is usually in the form of an ordinance or ordinances. Lines, grades, and the like are said to be established when the board of trustees, councilmen, or aldermen affirm certain defined lines and grades by enactment. An "established" line or grade controls all others within the city.

When the line of a street and its width and grades have been established, it devolves upon the city engineer to set proper monuments and bench marks so that other engineers, surveyors, and land owners may construct streets, curbs, fences (on property lines), buildings, bridges, and other permanent features in conformity therewith.

To accomplish this, center-lines and intersections must be carefully determined by survey, stone-bounds that are properly witnessed by tie lines must be set, and their position must be indicated correctly upon suitable maps. It is also expedient to chart profiles of principal streets, so that the ground surface and the established grade may be fully shown so that future

Figure 75.


N
16° E
W
E
Decl.
S
Grade 1%
Grade ½%
S B
80'
105'
100ft
15'
30'
6.0'
N 85° 0' W
S 85° 0' E
50'
N 5° 0' E

improvements both in the adjoining property and conformity with the plan as adopted and established by the governing body of the city may be assured.

(194) Let us now take a case of laying out a portion of a town-site, with streets, alleys, and city blocks. A portion is chosen embracing part of a street in the business section. The references are to Fig. 75.

(195) The main thoroughfare is a street 100 feet wide, measured from property line to property line, with sidewalks 10 ft. wide to the line of curb; the street is therefore 80 ft. wide between curbs. Crossing the main street are narrower streets, 60 ft. wide, with sidewalks 8 ft. wide and a distance between curbs of 44 ft. The blocks are 225' x 225'. An alley is out through the block parallel to the main street, 15' wide. The subdivision of the block into lots 25 ft. x 105 ft. is also shown. The cross streets are shown as intersecting the main street at right angles, which is the simplest method, as it is desirable for our purpose not to complicate the problem.

A line is run along the center-line of the main street, as shown by the dash-dot line; suitable monuments are placed at the mid points of intersecting streets. These may be either permanent ones, or temporary points to be located permanently later on, or perhaps defined by bearing and distance or other coordinates. Off-sets are next taken from the point of intersection on either side of both streets, in this case 30 ft. along the main street, 50 ft.

along the side street. The center lines of the side streets are now run at the proper bearing, and the mid-points of the alley ways, the location of the property corners, curb lines, and other points are staked out. Curbs are finished with rounded corners but in the survey the stakes are driven at the points of intersection at the corner.

When the town is laid out before being built upon, the system of streets, blocks, alleys, curb-lines, etc., can be surveyed with strict conformity with a uniform plan as shown. In such case, the lots also may be staked out, a tack-centered hub placed at each corner, witness or guard stakes should also be driven; on these should be placed the lot and block numbers to serve for identification in the field.

While the business portion of a city should have a more or less rectangular plan, the residence districts may conveniently conform with the topography of the region; the streets may even follow principal contours over hilly portions thereby conserving both grade and direction; the individual lots in such cases may be quite irregular in shape and size; here attention must be given to frontage and depth to afford a suitable building site with proper lawn and garden and space for necessary out-buildings.

Streets may intersect at all conceivable angles and turn upon regular curves both horizontal and vertical. Horizontal curves such as are required in railroad construction are commonly employed, but these are generally of simple kind, and can readily be laid out by the usual methods in such cases.

PROB. NO. 20. LOCATION OF A SIMPLE CURVE.

P-4

Station	Point	Descrip. of Curve	Total Deflect.	Observed Bearing	Remarks.
110					Error in dist. = 0.03 ft.
109					Error in angle = 0°01'
+90.00	P.T. ⊙	I = 22°30'	11°15'	N46°00'E	{B.S. on Sta. 106
108		L = 450.00	9°00'		{Vernier 4°00'
107		T = 228.01	6°30'	Diff. =	
106	⊙	R = 1146.28	4°00'	22°45'	
105			1°30'		{B.S. on P.C.
+40.00	P.C. ⊙	D = 5°00'	0°00'		{Vernier 0°00'
104					
103					
102				N23°15'E	

Equipment:-

Berger Transit. #9165L.
100-foot steel tape.
Line rod.
Reading Glass.
Chaining pins.
Plumb-bobs.
Hammer.
Stakes & hubs, etc.

Jan. 16, 1910.
Cloudy, showers.

Kirby, W.L. — Rod
Lakewood, N. — Rod
Milton, R. — Chain
Caine, N. — Chain

COMPUTATIONS.

$I = 22°30'$

$D = 5°00'$

$L = 100\frac{I}{D} = 100 \times \frac{22.5}{5} = 450.0$ ft.

$R = \frac{50}{\sin \frac{1}{2} D}$ = 1146.28 ft.

log 50 = 1.698970
colog sin 2°30' = 1.360328
log R = 3.059298

$T = R \tan \frac{1}{2} I$ = 228.01 ft.

log tan 11°15' = 9.298661
log T = 2.357959

First subchord length (nominal) = 60.00 ft.

First deflection angle = $\frac{60}{100} \times 2°30' = 1°30'$

True length first subchord = $100 \frac{\sin 0°45'}{\sin 2°30'}$ = 60.01 ft.

Deflection angle for full station = 2°30'

Last deflection angle = 11°15' = $\frac{1}{2} I$.

Last subchord (nominal) = 90.00 ft.

True length last subchord = 90.00 ft.

(196) The Simple Curve

In Fig. 76, AB and BD arethe center lines of two streets intersecting at B. It is desired to connect these by a curve, which should be tangent to both lines of roadway. The deflection angle EBD, called here the intersection angle, is measured. A suitable degree of curve is then determined and its radius computed.

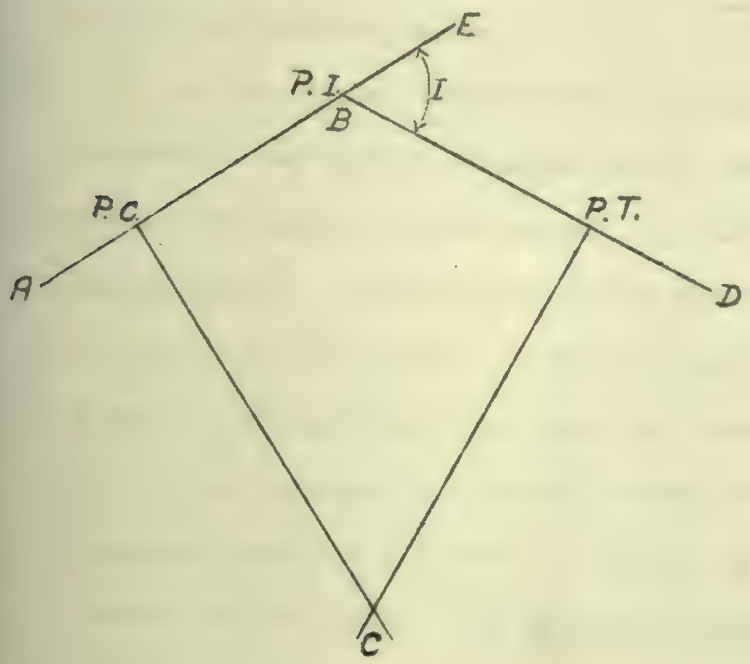


Figure 76.

To do this it is necessary to make certain assumptions. These are as follows:

1. The curve is laid out by chords of regular length, usually 100 ft. long.

2. The number of such chords, dependent upon the ratio of the intersection angle to the degree of curve, will govern the length of the curve, and hence the curve may begin at any point assumed on either tangent AB or BD. In railroad surveys, it is the practice to make each station on the line 100 feet and in most cases the point of beginning of a curve is not coincident with a

full station; therefore fractional chords are common. But as stationing is not practiced in city surveying, it is the practice to begin curves at the point of beginning and to close on the point of closing when convenient.

(197) In the figure, perpendiculars are drawn to the points of tangency of the curve joining AB and BD; thus the angle C, called the central angle, is constructed. It will be seen that angle I equals angle C. If we wish to make a given number of chords, n, then we divide the angle I (= C) by n which gives us the degrees of curve. Having chosen the degree of curve, with this consideration, it is necessary to know the points of tangency, i.e. the tangent distance of P.C. and P.T. (point of curve and point of tangent, respectively). The geometric relations of these points are as follows:

$$\frac{I}{n} = \text{degree of curve (D)}$$

$$\text{Radius of curve} = \frac{100}{2} \div \sin \frac{D}{2} = \frac{50}{\sin} \; 1/2 \; D$$

$$\text{Tangent Distance} = R \cdot \tan \frac{I}{2} \; (= R \cdot \tan \frac{C}{2})$$

We may now locate P.C. and P.T.; this is done and tack centered hubs are driven at both points.

To lay off the 100 foot chords from P.C. to P.T., it is necessary to know the amount that each chord deflects from the tangent AB. Draw the line P.C. to P.T.; this is known as the long chord. The angle formed by the tangent and long chord intersecting

at P.C. or P.T. is equal to 1/2 I. This angle, divided by the number of chords in the curve, gives the value of each deflection from the tangent. Hence, if n equals the number of chords, and I equals the intersection angle, then the deflection angle d is found from the formula $d = \frac{I}{2n}$. With this angle laid off from the tangent, the first chord of 100 feet is measured from P.C.; then with double the deflection angle d the second chord is laid off, and so on until the curve is completed.

(198) Let us take a special example. Suppose that two roadways intersect at an angle of 70°; it is desired to lay out a curve of say five chords and therefore the degree of curve selected is $14° - \frac{I}{D} = n = \frac{70}{14} = 5$. The radius of a 14° curve will be:

$$R = \frac{50}{\sin 7°} = \frac{50}{0.1219} = 410.17 \text{ ft.}$$

Tangent distance then is:

$$T = R \tan 1/2\ I = 410.17 \times 0.7002 = 287.2 \text{ ft.}$$

A deflection angle of 7° is obtained from

$$d = \frac{I}{2n} = \frac{70}{2 \times 5} = 7°$$

To locate the curve in the field proceed as follows:

Measure off the tangent distance, 287.2 ft. from P.I. to P.C. and P.T. Set up transit at P.C. With vernier set at 0°, upper motion clamped, sight upon P.I., clamp lower motion, bisect P.I. by use of tangent screw. Turn off the deflection angle, 7°, and, with 100 ft. tape, measure the first chord from P.C., lining in head chainman with the transit telescope. Drive hub and set tack center.

Turn off $2\frac{I}{2n} = 14°$, measure the second chord from end of the first chord, lining in with transit telescope as before, and proceed thus until the five chords are staked out in the field. As a check set up transit at P.T. and check back on the angles; these should be the same in reverse - 7°, 14°, 21°, etc. The curves of roadways may be

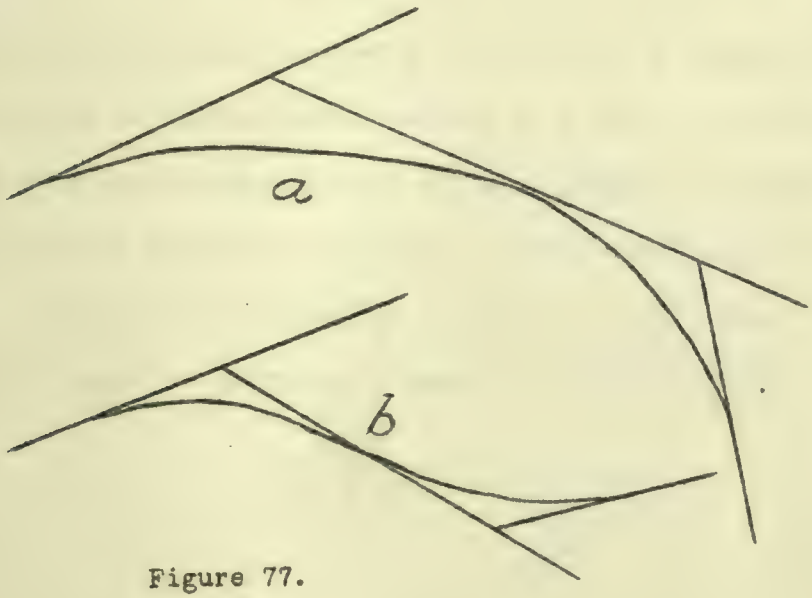
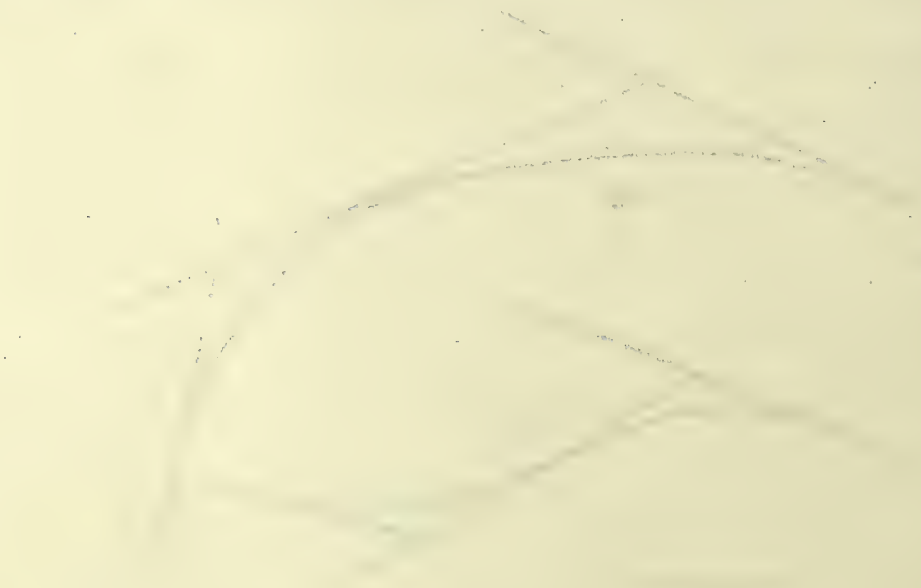


Figure 77.

compounded, as in case of several tangents intersecting at different angles, the deflections being either in the same direction or in reverse as shown in Fig. 77, a and b. The adjustment of tangent distances is a prior consideration to degree of curve which may be deduced from the formulas given above.

$$\text{Tan. Dist.} = R \cdot \tan\frac{I}{2} \quad \therefore R = \frac{\text{Tan. Dist.}}{\text{Tan}\frac{I}{2}}, \text{ and}$$

$$\text{since } R = \frac{50}{\sin\frac{D}{2}} \quad \therefore \sin\frac{D}{2} = \frac{50}{R} \text{ or } \sin\frac{D}{2} = \frac{50 \tan\frac{I}{2}}{\text{Tan. Dist.}}$$



References:

Tracy	pp. 188-190
Raymond	pp. 230-243
Johnson	pp. 356-370
Breed & Hosmer	pp. 251-275

Problem to accompany Assignment 26

Two center lines of streets intersect at an angle of 60°; it is required to connect these streets by a curved roadway having a degree of curve giving six full 100 foot chords. Find the degree of curve, radius of curvature, tangent distence, deflection angles. Supposing the deflection to be to the left, make the paper location of the curve on a scale of 1 inch = 100 ft.

Foreword

A continuation of city surveying with special consideration of coordinate systems, irregular subdivisions, andsubdivisions of additions to townships, will be the work of this assignment.

(199) Street Grades

While street grades depend largely upon the topography of the town-site, it is always desirable to reduce these as nearly to level as is consistent with good drainage. Where the land is especially irregular and hilly, streets should conform as nearly as possible to the relief features in order to preserve beauty of landscape. Yet the whole must be treated in such a way as to afford convenient and orderly arrangements of blocks and streets so that the city may possess proper order in both its business and its residential districts. A strict adherence to rectangular forms, straight streets, and uniform blocks may prove difficult, but as a city grows in size and business importance, these things become of greater and greater significance. Hence in planning cities consideration must be given to such matters, for otherwise many difficult problems, such as the widening and straightening of streets, changing of grades, etc., will be likely to arise in the future. Such changes are attended with great expense and are often the cause of much troublesome bickering and litigation.

So far as possible street grades should conform to the grades of property along the street; otherwise either the street

work becomes too costly or much filling or leveling is required to bring the property into grade suitable to the roadway. The gradient of the street should be established with regard to the many matters above referred to, and improvements relative to the construction and maintenance of curbs, to drainage, and to roadway should be conducted in accordance with them.

(200) In the case of irregular shaped blocks, it is often uncertain whether the lot lines should run perpendicular to one street or another and also whether for the sake of uniformity the lots should extend longitudinally in one direction or another.

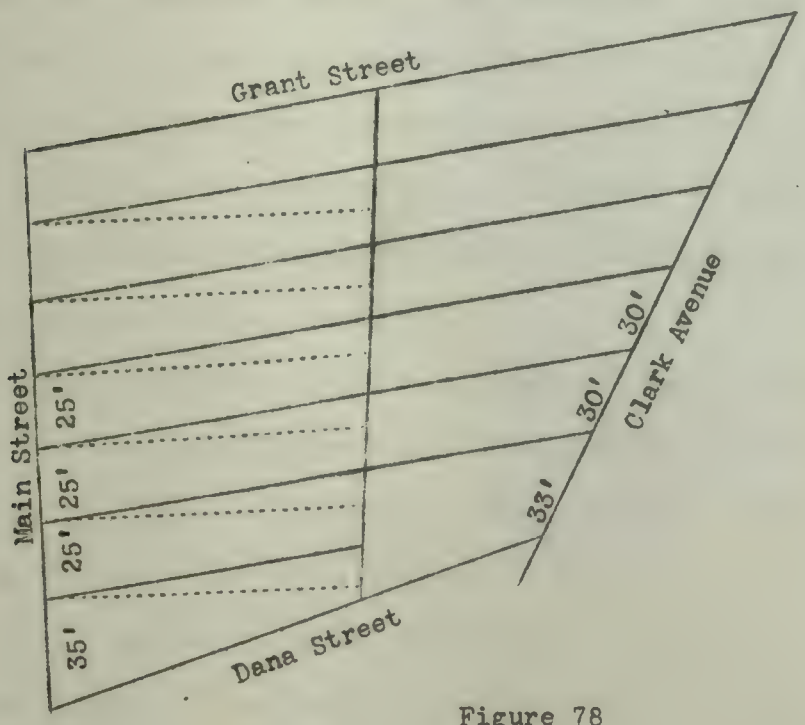
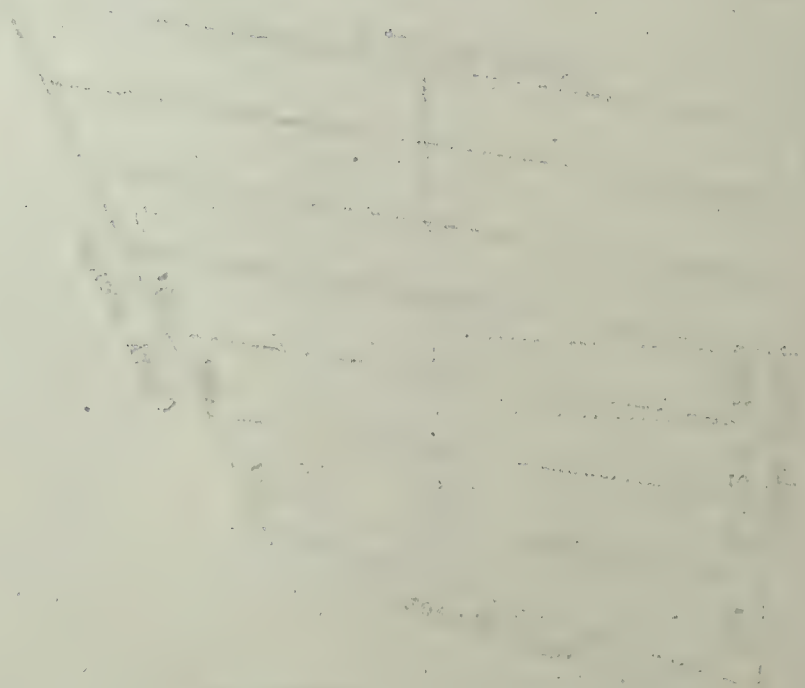


Figure 78

In Fig. 78 it is of course desirable that the frontage of blocks shall be upon Main St. (here supposed to be a principal business street), but it is also desirable that the lot lines shall be perpendicular to the line of Main St. The oblique angles are



not objectionable provided the obliquity is not great, as shown on Clark Ave.; still, by adjusting the boundaries, as shown by the dotted division lines, the lines may be rectified so as to preserve at the same time the uniform width of lots along Main St. On Clark Ave. the oblique angles are not so objectionable, since this is a residence street. The houses are set back from the street line andthe sidewalks are made parallel to the street.line.

Much ingenuity and some taste may be exercised in subdivision of large irregular tracts in residence additions, so that roads, parking, and lawns may present a picturesque and pleasing appearance. Where such a plan does not involve great waste of land but gives convenience of arrangement for building sites, streets, etc., it is preferable to the rectangular and conventional regularity of city lots that are so common because they economize space and afford convenient street plans.

(201) Plans for sewers giving their location both in plan and profile should be carefully made both to assist in estimates, contracts, and actual work of construction and also to facilitate the making of alterations and repairs.

Water mains, gas mains, and electric conduits should also be carefully surveyed and mapped for like reasons. Cities are now generally requiring that all such mains be placed not in the roadway but in the parking within the lines of curb or through "easements" conceded by property owners along the rear lines of adjoining holdings; alleys in some instances take the place of such "easements" and are thus made a public care.

(202) To provide suitable drainage, streets are built higher at center than at the curb on either side. This raising at center is called "crowning." Generally it is sufficient to make the center as high as the top of the curb and since the crowning is always determined by the elevation of the curb, it is required that appropriate grade stakes be put in for setting curbs. This is done by setting a convenient bench mark, which may be brought up by differential leveling from some other bench mark or datum. When the relation of the curb profile to this new B. M. has been found, grade stakes are set for curbs. Stakes should be driven firmly into the ground along the line of curb and the actual elevation marked by a line upon the stake in order that workmen, who are not engineers, may work to the line designated.

A cross section of a street from curb to curb is shown in Fig. 79. Here the special case where one curb is higher than the other is chosen, for the case where both curbs are at the same height is easily understood.

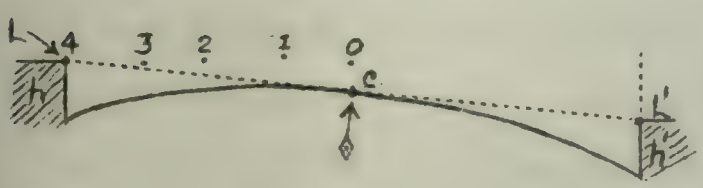


Figure 79

The crowning at C is made equal to the height of curb at h (the higher side). The form of the curve of the finished street is generally parabolic and hence at points 1, 2, 3, 4 the depth below the straight line L L' is as the square of the distance from C on either side. The depth of gutters at each side is determined by the quantity

of surface drainage to be carried off, and hence the height of C varies with this condition.

(203) Widening and straightening of streets in cities calls for careful surveys first to determine the old lines and then for the several plans to bring about the most desirable revision. All such changes are attended by great expense and often entail hardships upon property owners. Where practicable, therefore, the most economical course should be followed.

(204) Monuments of surveys, benchmarks, etc., are made numerous in cities owing to their frequent use and they should be placed at convenient points for reference. These can seldom be placed in the roadways as they are disturbed by traffic, and are likely to be changed or wholly obliterated by street work that is frequent even in crowded thoroughfares. Hence all monuments and bench marks are best located on curbs, corners of buildings, or by specially constructed structures not in but on or near the street. Often a mark properly referenced, cut into a curbstone may serve the double purpose of a stone bound and a benchmark. Such marks are inexpensive and can be plentifully established throughout a city, especially in the districts where they are much needed.

(205) The method of laying out of streets and city blocks as heretofore described by street lines properly monumented and by a general system of rectangular blocks is by far the most common, but a coordinate system is by far the more desireable from many considerations.

The co-ordinate system starts with an initial point assumed usually upon some established meridian and with the two axes, Y in conformity with this meridian, and X at right angles to this extending in an east and west direction. The corners of all blocks, lots, centers of streets, and positions of public buildings are therefore located with respect to the intersection of these axes as X's and Y's. The relocation of any corner, disturbed by any cause can readily be made, or monuments restored or established anew by resort to measurements from these axes. By such a system the work of the surveyor is at once simplified and a higher degree of precision made possible. Such systems are best when made independent of curvature and convergence of meridians, all north and south lines being made parallel to the Y axis, all east and west lines parallel to the X axis.

Triangulation systems are also in use in some large cities. These systems require the establishment of a base line determined in position by careful astronomical observation and measured with extreme accuracy and properly monumented. Then as in geodetic surveying, a net of triangles beginning upon this base is determined, their corners are permanently marked throughout the territory, and the primary survey proceeds in the usual manner. All angles are measured either with a direction theodolite or by the method of repetition with a precise transit capable of reading to 20", or better, to 10".

The angles are adjusted for each triangle and any small dis-

crepancy in the total interior measure, which should of course be 180°, is distributed. The sides of the triangles are then computed, the lengths of all lines being checked as the computation proceeds.

(206) As before stated the instruments employed in city surveying should be of the more precise kind. A tape should be graduated to hundredths throughout and compared with a known standard. It is best to know the normal tension at which the tape may be used; and as temperatures may vary greatly from time to time, the temperature should be taken with each measurement requiring great accuracy. Generally it will be better to take tape measurements along the slopes and either determine the angle of slope or the difference in elevation between the ends of any segment of a line where the slope changes. The usual corrections are then readily and accurately made for obtaining the horizontal distance. It is not required in taking the difference in elevation to use an engineer's level, as a transit with level on telescope may be conveniently substituted, and in many cases differential leveling with the hand level or measuring angle of slope by means of the clinometer will suffice.

The surveyor on city work should recognize that there should be different degrees of accuracy sought in the various kinds of measurement. In all primary work such as setting of monuments, stone-bounds, benchmarks, etc., measurements should be taken with great care, carefully corrected for standard temperature, slope and so forth, and each monument should be checked usually by a second or third set of measurements conducted by parties of a different

personnel. All monuments and stone bounds should be accurately referenced, so that they may be properly restored if disturbed. Since such points are arbitrarily set in the primary survey it is well to make these fall at an even foot-mark where practicable, but the measurements taken in the establishing of them should have regard to the nearest hundredth or possibly thousandth of a foot. Street lines, on which many blocks and lots are likely to abut, should be run in conformity with a fixed meridian or its bearing therewith should be carefully determined to within a few seconds of arc. This bearing may be secured by use of a precision instrument, a direction theodolite or a good transit. If a transit is used, the method of reading angles should be by repetition, six repetitions normal and six inverted and by reading on varying parts of the limb in order to eliminate errors in eccentricity or graduation.

Differential leveling for establishing bench marks should be conducted forward from a permanent datum, and rod readings made to the nearest thousandth of a foot. The level rod should be accurately graduated and compared with a standard tape, or carefully calibrated. Readings should, therefore, be made upon targets and with the use of a rod level or other plumbing device. The index error of the rod vernier should also be determined and the target setting for long rod checked as well. In short, all possibilities of instrumental errors should be eliminated.

(207) The extreme accuracy aimed at in work described above is not required in measurements of secondary importance, such as setting of curb lines and grades, placing of fence lines, surveys for surface and

underground drainage, sewers, water mains, gas mains, electric conduits, etc. In such surveys an accuracy to the nearest tenth of a foot and lines run practically parallel to existing lines are sufficient. Time and labor on making refined measurements in these latter cases but add to expense without securing a requisite degree of precision.

In general, city surveying requires few data obtained in the field, but complete notes and clear accurate sketches must accompany all field data which also should be immediately recorded in original field books. These field books also should be labeled and properly indexed, in order that the data relating to any given survey may readily and accurately be found.

Office work consisting in the computations from observed and recorded data, making of maps, detailed sketches, etc., should be carried forward as the work progresses and surveys are correlated with other work of record.

References:

Johnson, pp. 356-383

Tracy, p. 188

Raymond, pp. 230-236

Breed & Hosmer, pp. 251-286, Vol. I.

Problems

1. The line of Main Street in the town of A bears N 17°30' E of true north; First and Second Streets intersect Main Street at an angle of 87°10'. What is the bearing of First and Second Street lines with true north?

2. If the lot lines on Main Street are perpendicular to the line of Main Street, what is the bearing of these lines?

3. Sketch Main, First and Second Streets; place arrows on sketch, showing both true north, magnetic north (declination, 12°15' East) and write the respective bearings on the lines so drawn.

UNIVERSITY OF CALIFORNIA EXTENSION DIVISION

Correspondence Courses

Surveying-1B Assignment 28 Mr. Swafford

RAILROAD SURVEYING

Foreword

In the present course it is the purpose to give only a very elementary treatment of the subject, Railroad Surveying, as this branch of the work is of a highly complex nature, presenting many problems requiring the highest skill of the engineer.

The work in railroad surveying is divided into several parts, the order of progress depending upon the stages of such work. The three stages here considered will embrace (1) Reconnaissance Survey, (2) Preliminary Survey, (3) Location.

(208) The Reconnaissance Survey

When it has been determined to connect two or several localities by a line of railroad the first work devolving upon the engineer is to reconnoiter the territory to be traversed and to gain by this means a knowledge of the character of the country - its plains and hills, its water courses, timber, soils, rocks, etc. This is done in order to determine the most desirable and likewise the most feasible or the least expensive route to be followed in order to carry out the purpose of the road. It is often necessary to examine not one, but two or more different routes by which this purpose may be accomplished. This part of the work constitutes the reconnaissance survey which consists, in short, in discovering the route of the proposed railroad.

To do this the chief engineer of the road, equipped with the necessary instruments and accompanied by one or more aids, traverses the territory choosing a route, or several routes. He observes the lay and character of the land more or less minutely. He should travel slowly either on foot, where wagon roads are wanting, or by light conveyance on such roads as parallel the desired route. In other words, he must actually traverse the lines it is intended to follow.

He should be equipped with a compass, a hand level (which may also combine a clinometer) a field glass, and a pocket tape. A 50 foot cloth tape is a convenient one to handle and is sufficiently accurate. If a wagon is used, a rodometer attached to one wheel will be found excellent for determining distances traveled by road. An aneroid barometer will likewise be useful in determining elevations from time to time. The intermediate elevations may be determined by observation with the hand level or clinometer.

Over some portions of the route it may be expedient to make topographic maps by plane-table. For regions where the Geological Surveys of the country have been made the very excellent contour maps published by the Government Bureau may be procured. Such maps should be used not in lieu of a personal examiniation of the ground itself, but only as an excellent aid to a better understanding of the nature of the territory traversed.

The engineer should make copious notes, collect much desirable data, and draw frequent sketches to assist his memory. In fact he should trust little to memory and <u>never</u> where the data are obtainable

and the record can be made. His survey, when completed, should enable him to visualize the entire route, section by section, in such manner as to draw in imagination and afterwards in reality any portion of the route selected; and to indicate thereon practicable grades, location of bridges, cuts, or tunnels, and the deflections of his road from point to point.

Not always does he choose the most direct route between two given points. Such a course often entails too great an expense in construction, as it may require bridging a stream several times, or the making of costly cuts or tunnels. Often an engineer finds that by keeping to the comparatively level lands along the course of a stream, even when the sinuosity of the stream is great, he is able to construct his road at smaller cost, although it might be desirable to straighten the road subsequently. Along the broad alluvial plains of streams the cost of construction is usually at the minimum, as it requires at most only moderate embankments that shall keep a road bed well above any possible inundation in time of freshets. Moreover, the character of the soil lends itself to cheapness in construction while the gradient is always gradual, approaching or falling away from a watershed divide by easy stages. A wide detour is often made necessary to avoid bridging or cuts that would otherwise be necessary.

Another advantageous location for a road is found in following hill-side formation as the materials cut from the upper portion are at hand to make the fill upon the lower portion. Ledges are often found in some localities that further contribute to cheapening

construction. Often, too, by means of an occasional trestle that carries the road over a small ravine one series of skirting contours after another may be utilized, and thus afford an economical route.

All such matters as those mentioned above, which the engineer encounters upon his trip over the proposed route, are carefully noted, lines are drawn, elevations taken, and sketches made to elucidate the conditions. These are then embodied in a report to the railroad's board of directors. Before completing his report, if time and funds permit, the engineer should retrace his journey noting alternative routes or plans, verifying or correcting observations, and choosing perhaps the more desirable of two routes.

Out of this mass of information a plan is formulated and a proposed route finally determined upon, which shall guide the engineer in staking out upon the ground, by tangents and deflections and by a system of profiles more or less complete, the defined route of the road. This survey constitutes the preliminary survey of the road.

(209) The Preliminary Survey

This survey is under the direction of a chief of staff who proceeds to set out a series of straight lines of varying length following the route already determined in the reconnaissance survey. To do this the engineer consults always the notes, maps, and indicated route formerly prepared, making such changes as may be necessitated by conditions encountered. The passing of a serious obstacle or the choosing of a more advantageous course may of course necessitate the alteration of the route previously selected, but

such alteration should not be made for light or transient reasons. All data of the reconnaissance survey are, however, subject to liberal interpretation, and a line may be made to deflect left instead of right or be terminated or prolonged to suit the careful exercise of judgment on the part of the chief carrying out this preliminary survey. Lines and grades actually laid out in the preliminary survey are not hard and fast lines, for they may themselves be revised when the location is in progress; but the preliminary survey should in the main be conducted very much as though such were the case. The preliminary line is therefore staked out upon the ground. An accurate map to scale and a set of notes complete in every detail are made which, of course, must be in perfect agreement.

The preliminary survey parties are three, all under the direction of the chief engineer. They consist of a transit party, a level party, and a topographic party. In the first are required a transit man, two rodmen, an axe-man, and a stake-man. In case the compass is used instead of the transit the rear rodman may be dispensed with. The instrument man of course handles the transit (or compass) and conducts the work of his party under the general direction of the chief of staff. The level party is composed of a level-man and one or two rodmen who follow the transit party as closely as possible and at the close of the day are perhaps only a few rods in rear of the transit party. One or two topographers can take care of their portion of the work, but the topographers should keep well up also with the levelers, and transit party, as it is

[illegible] examination should not be made for rights or easements [illegible]

[illegible] record [illegible] and a time may be made to continue [illegible] stood of trial or be terminated or dismissed to omit the criminal [illegible] evidence of documents or the part of the court concerned one thing [illegible] particularly effect. Times and dates necessary [illegible] in the [illegible] [illegible] merely [illegible] and [illegible] [illegible] [illegible]

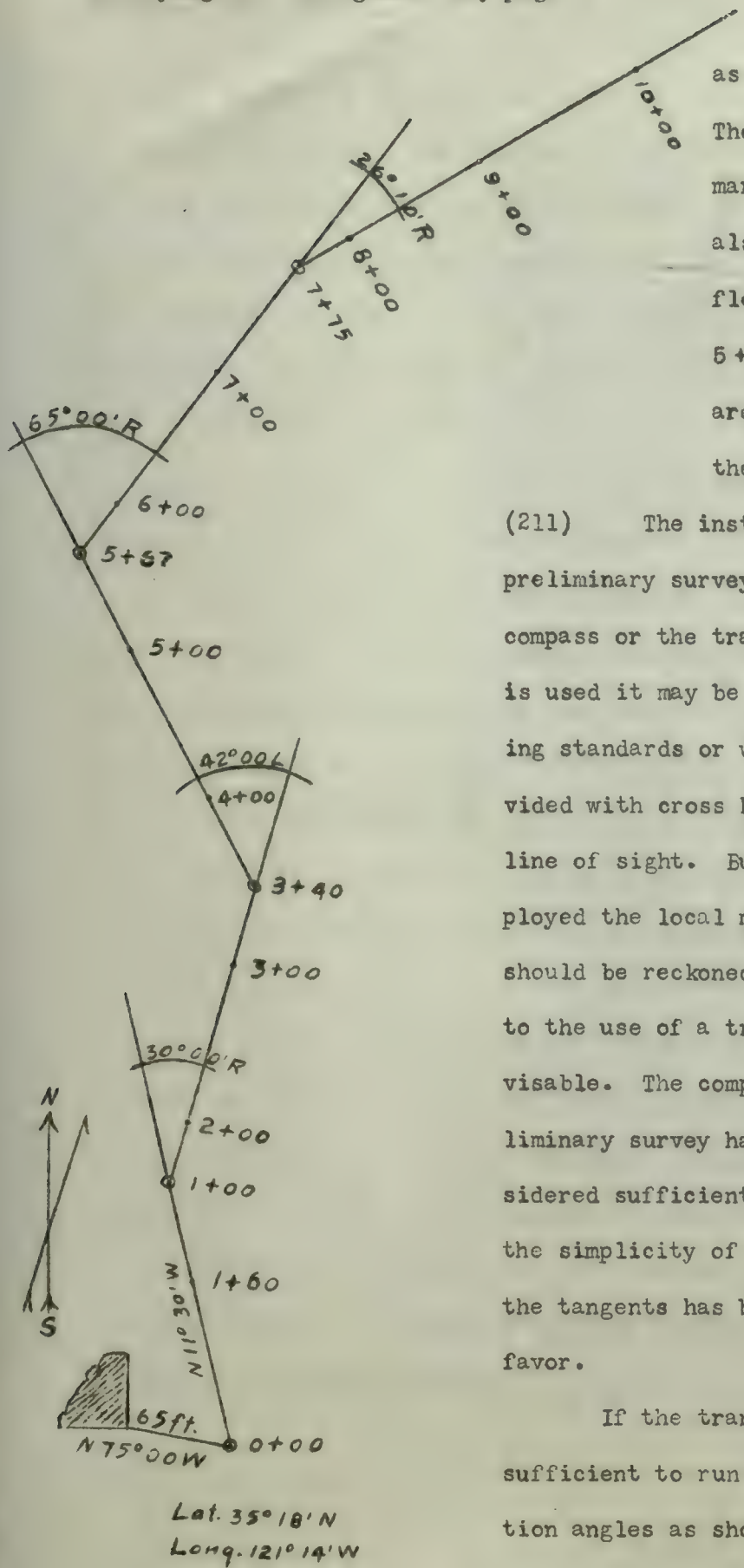
necessary frequently to procure requisite data from these other parties.

Having exactly determined the point of beginning of the road about to be projected, the chief of staff establishes this point by setting a permanent monument of suitable character. The latitude and longitude of this point is carefully determined by astronomical observations, these being entered in the notes of the preliminary survey. The point is also tied to two, or better, three fixed land marks of more or less permanent character, the bearing and distance of which are likewise recorded. This point is designated as the zero point of the line and is referred to as 0+00 (zero plus naught naught).

The subsequent stations, successively 100, 200, 300 feet distant and so forth, on the final "location" are then designated 1+00, 2+00, 3+00, etc.; other points on the line are called pluses, as at B, 1+60; at c, 3+40; etc.

(210) For purposes of illustration the manner of ranging out a railroad is given in Figure 80 given on the following page.

Beginning at atation 0+00, the latitude and longitude of the station having been determined and the point tied to a permanent fixed point by bearing and distance, the line is ranged N 11° 30' W 160 ft.; then the line deflects to right 30° 00' to a point 340 ft. from point of beginning; thence 42° 00' L to 567 ft.; then 65° 00' R to 775 ft.; thence 26° 10' R to 1000 ft. (The distances are made quite short to condense the diagram, but the principle is the same



as for longer tangents). The full stations are marked upon the line as also the points of deflection as 1+60, 3+40, 5+67, 7+75, and these are always measured from the initial point.

(211) The instrument used in the preliminary survey may be either the compass or the transit. If the compass is used it may be equipped with sighting standards or with a telescope provided with cross hairs for giving the line of sight. But if a compass is employed the local magnetic attraction should be reckoned with, and resort to the use of a transit may prove advisable. The compass on the preliminary survey has usually been considered sufficiently accurate, while the simplicity of reading bearings of the tangents has brought it much into favor.

If the transit is used, it is sufficient to run the line by deflection angles as shown above, Fig. 80.

The first segment of the line is ranged out, either with or without the use of the transit; the transit is then set upoon a forward point of the segment, a back sight with telescope inverted is taken upon the former station, the telescope inverted to normal and the line continued in line with the rear segment, and this process is continued to the point where the course of the road may turn to the right or left (i.e., deflect).

When the point of deflection is reached the transit is set up over this point, the upper plate is clamped with the A verneir of 0°, and with the telescope plunged a backward sight is taken to any convenient point on the last preceding segment; the lower plate is now clamped and the telescope is then turned to normal, which places the line of sight in line with the continuation of the last segment (or tangent of the road). If now the upper plate be unclamped and a sight (forward) be taken on a point on the next succeeding tangent, the vernier plate of the transit will read the deflection angle at the point occupied.

(212) Note keeping on the preliminary survey is of especial importance; all data that can in any way contribute to clearness in determining the next important step, towit, the paper location of the road, must be gathered in systematic arrangement and neat form.

Specimen notes are shown on the accompanying sheet.

The student is referred to Searles & Ives Handbook of Field Practice in Railroad Surveying.

Questions to Assignment 28

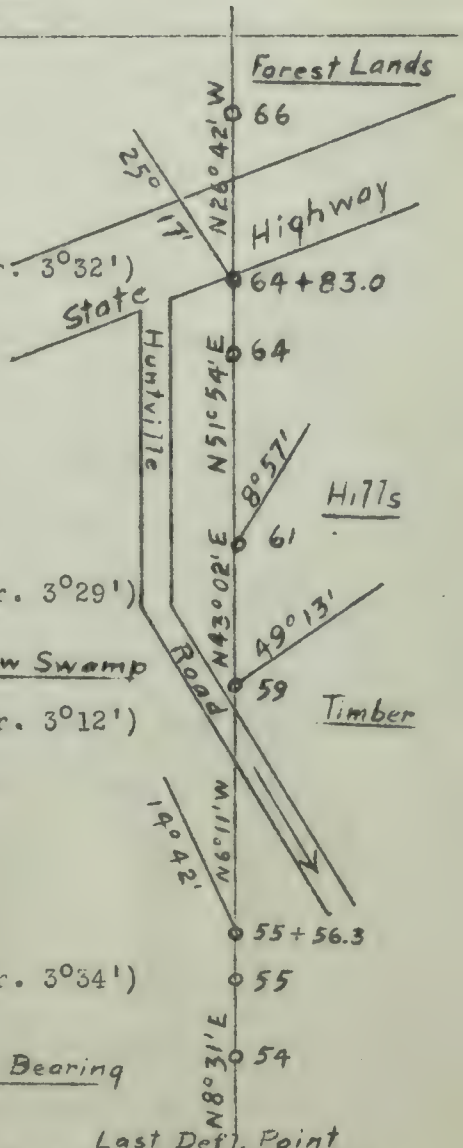
1. State the general outline of work of the chief engineer on a railroad reconnaissance survey.

2. What are the data required in the preliminary survey?

3. Define deflection angle, tangent, and bearing.

Notes of R. R. Preliminary Survey

Sta.	Defl.	Magnetic Ang.	Magnetic B.S.	Magnetic F.S	Bearing	
66						
65						
0	25°17'L	25°20'L	N48°30'E	N23°10'E	N26°42'E	(Var. 3°32')
+83						
64						
63						
62						
61	8°57'R	9°00'R	N39°30'E	N48°30'E	N51°59'E	(Var. 3°29')
60						
59	49°13'R	49°10'R	N 9°20'W	N39°50'E	N43°02'E	(Var. 3°12')
58						
57						
56						
56.3	14°42'L	14°40'L	N 4°55'E	N 9°45'W	N 6°11'W	(Var. 3°34')
55						
54		Fig. 81			(N8°31'E)	



Note: The notes are arranged to read from the bottom of page upward, as shown by numbering of stations; the notes here given are a continuation from a preceding page. Sketch on the right gives principal data to assist in mapping the "Preliminary" and also the "Paper Location". Important topographic features are shown, but minor details are not necessary, and hence are omitted from sketch.

UNIVERSITY OF CALIFORNIA EXTENSION DIVISION

Correspondence Courses

Surveying 1B Assignment 29 Swafford

Railroad Surveying - Tangents and Curves

Foreword

In this assignment an extended consideration of the simple curve as used in railroad surveys will be made. The student is directed to the latter part of Assignment 26 where the use of the simple curve on streets in City Surveying is treated.

(213) Tangents and Curves

A line of railroad is essentially a series of straight lines, called tangents, and curves - the latter connecting the former in continuous sequence.

In the preliminary survey a succession of straight lines of varying length which intersected with each other from point to point along the route, were staked out. The roadbed and especially the tracks could not be of use in carrying trains over such a course, and it becomes necessary to connect the straight segments of road by curves of varying degree of curvature. These curves, circular in form, are made tangent to the two lines of direction, and are of arbitrary radius (or degree).

For any given angle of deflection of the line, the degree of the curve is chosen, usually from several considerations. Matters influencing the choice of degree of curve are :- proximity to other curves, or distance apart of points of intersection (deflection vertices), nature of grades encountered on the line or in proximity to the curve, nature of earth materials on which

the curve is to be constructed, approaches to bridges and tunnels, even the procurement of right of way, and the speed of trains operating over the curve. A small degree of curve is always to be sought, all things considered. Beyond such matters the choice of a degree of curve is rather an arbitrary matter left to the judgment of the engineer in charge. The simplicity of the computations should not be wholly overlooked in the selection; generally the larger the deflection of the tangents (i.e. the larger the intersection angle) the smaller the degree of curve; hence, the more gradual the curve connecting straight lines.

For example, if the intersection of tangents is 40°, a 5° curve would give 8 chords (or hundred-foot stations) on the curve, while a 10° curve would give but four stations. The former would permit trains to move at a speed of say 40 miles per hour, the latter at only 25 miles per hour. In addition the work done on the former would be much less, the wear on roadbed and rails, as also the wear on rolling stock would also be less for the 5° curve than for the 10° curve.

Referring to Fig. 82, AQ and MB are tangents intersecting at M. QMB is the intersection angle I (= Central angle C). If now a curve ANB is laid out tangent to AQ at A and to MB at B, AM and MB are called the tangent distances from the points of intersection (P.I.); A is the point of curve (P.C.) and B is the point of tangent (P.T.). Radii from P.C. and P.T. meet at C, the center of curvature. C is the central angle.

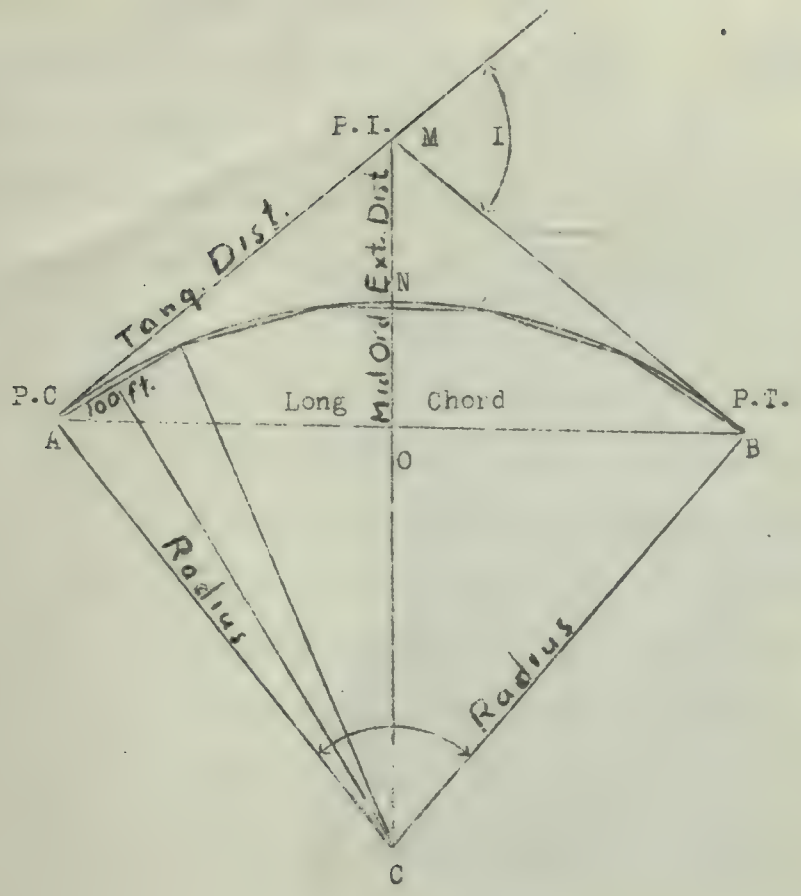
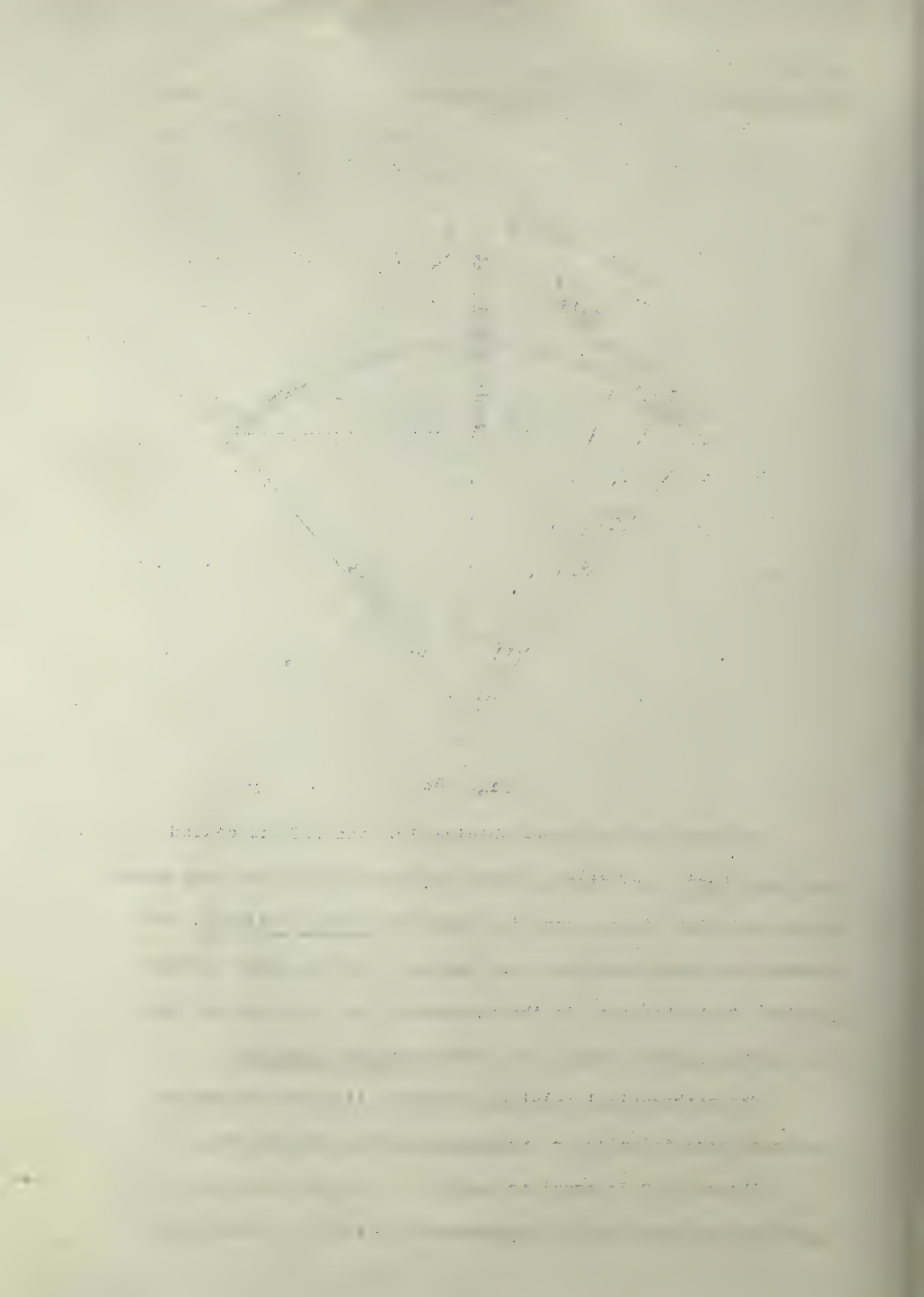


Fig. 82

The chord of the curve joining P.C. and P.T. is called the long chord. The distance from the mid-point of the long chord to the midpoint of the curve is called the middle ordinate; the distance MN from the midpoint of the curve to the point of intersection of the tangents is the ex-secant (i.e. the external segment of the secant), C.M., called the external distance.

The mathematical relations of these lines and angles are expressed geometrically, or trigonometrically. as follows:

The angle C is equal to angle I. I is the supplement of angle AMB and angle AMB is supplemental to angle C; hence C = I.



Angle MAB (or MBA) is equal to one-half of angle I. Angle I is equal to angle A, plus angle B, and angle A equals angle B; hence angle A = $\frac{I}{2}$, angle B = $\frac{I}{2}$.

Tangent Distance is equal to radius of the curve multiplied by the tangent of one-half angle C (or one half angle I).
AM = AC tan C/2.

Long chord is equal to twice the product of the radius of the curve into the sine of one half C. L.C. = 2R sin C/2. (Since C = I, I may be substituted for C in the last two formulae.)

The middle Ordinate is equal to the radius diminished by the radius into the cosine of one half C (= $\frac{I}{2}$). Mid. Ord. = R - R cos $\frac{I}{2}$, which reduces to Mid. Ord = R (1 - cos $\frac{I}{2}$) = R vers $\frac{I}{2}$.

The external distance is equal to the square root of the tangent squared plus the radius squared diminished by the radius.
Ex. Dist. = $\sqrt{\text{Tan}^2 + R^2} - R$

The <u>degree of curve</u> is defined as the angle at the center (or its arc) subtended by a chord of 100 ft.

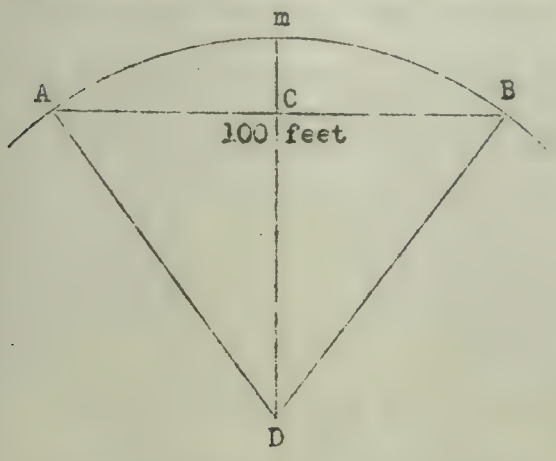


Fig. 83

In Fig. 83, AB, a chord of 100 ft. subtends the angle at C, or arc AmB; AC and BC are radii of this arc; by trigonometry:
AB : AD :: 2 sin D/2 : r,
r being the radius of unit circle. AB is 100.

Substituting $100/R = \frac{2 \sin D/2}{1}$; dividing by 2, $50/R = \sin D/2$; solving for R, $R = \frac{50}{\sin D/2}$.

There will be as many chords of 100 ft. in the curve, P.C. to P.T. as the degree of curve is contained times in the central angle. This may be expressed by the equation $N = \frac{C}{D} (= \frac{I}{D})$, where N is the number of 100 foot chords in the curve, I is the intersection angle, and D is the degree of curve.

The number of chords are, therefore, inversely proportional to the degree of curve; therefore the length of the curve from P.C. to P.T., the tangent distance, and the radius of the curve increase as the degree of curve decreases. It follows then, that as the degree of curve for any given intersection angle diminishes the flatter the curve becomes (or more nearly approaches a straight line).

The character of curve may be readily determined, therefore, by the number of stations on the curve; the radius of the curve, the tangent distance, the length of the curve (from P.C. to P.T.), and the long chord increase with the number of stations (i.e. the number of chords) and therefore inversely as the degree of curve.

It is the common practice to choose the degree of curve, as that for any given intersection angle determines the other elements of the curve. Some engineers choose a radius of curvature; and in some special cases it is expedient to assume the tangent distance, as the point of beginning of the curve or its relation to bridges, tunnels, etc., may make it desirable that the curve should begin or end at a definite point.

(214) The Deflection Angle

This is the angle which the chord makes with the tangent and is equal to one-half the intersection angle divided by the number of chords in the curve. In Fig. 84 is shown a curve of four stations (each 100 ft.).

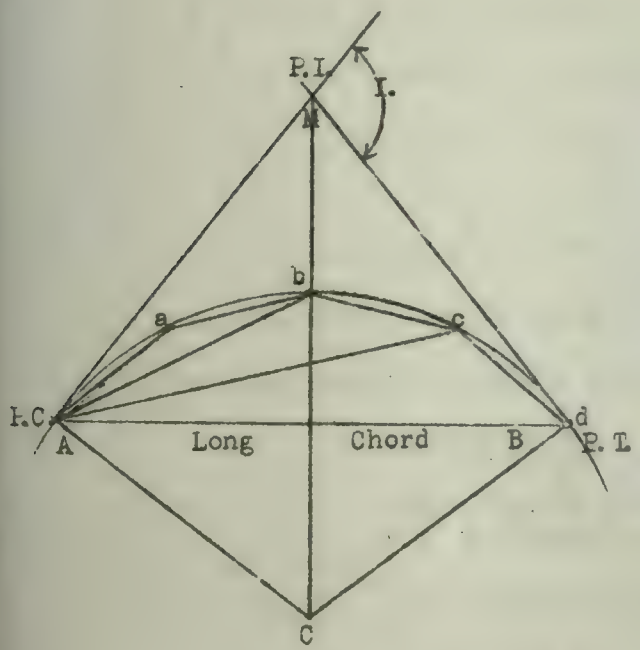


Figure 84

The angle BAM, which is the angle formed by the long chord and the tangent, is equal to one half I. MAa, MAb, MAc, MAd, are the departures of the lines Aa, Ab, Ac, Ad, from the tangent AM. The deflection angle is here I/2 ÷ n; since n in this case is 4, then the deflection angle is 1/4 of I/2 ($= \frac{I}{2n} = \frac{I}{8}$). The deflection angle is used in setting off the curve in the field. A transit is set up at P.C. sighted on P.I.; the deflection is then turned off on the limb I/2n, I/n, 3I/2n, 2I/n, the deflection angle being added successively to each preceding angle.

(215) We give in the following pages a few examples to illustrate and show the methods of computation for the several elements of simple curves.

1. Given the intersection angle, 60°; it is desired to locate 5 full stations on the curve which begins at the P.C. What is the degree of curve?

Solution: $N = I/D$
$D = I/N$
$D = 60°/5 = 12°$ ans.

2. If the intersection angle is 60°, the degree of curve 6°, what will be the length of the curve?

Solution: Length = 100N
$N = I/D$
$N = 60/6 = 10$
$\therefore$ Length = 100 x 10 = 1000 ft. ans.

3. The intersection angle is 70°; find the radius of a 5° curve and the number of chords in the curve.

$$R = 50/\sin \frac{D}{2}$$
$= 50 \div \sin 2° 30'$
$= 50 \div 0.04362 = 1146.28$ ft. ans.

4. In the above example what is the tangent distance?

Tan. Dist. = R. tan I/2
= 1146.28 x tan. 35°
= 1146.28 x 0.70021 = 902.64 ft. ans. X

5. For the same intersection angle and degree of curve what is (a) the number of full stations on the curve? (b) the deflection angle? (c) the long chord?

(a) $N = I/D$
$= 70°/5° = 14$ ans.

(b) $d = \frac{I/2}{N}$
$= I/2N$
$= 70°/28 = 2° 30'$ ans.

(c) L.C. = 2R. sin I/2
= 2 x 1146.28 x sin 35°
= 2 x 1146.28 x 0.57358 = 1314.97 ans.

The last solution is conveniently made by use of logarithms:

log 2 = 0.30103
" 1146.28 = 3.05929
" sin 35° = 9.75859-10
" Product = 13.11891-10 ∴ anti-log = 1314.94 ans.

6. The external distance (a) and the middle ordinate (b) may also be computed for the same curve.

(a) Ex. Dist. = $\sqrt{\tan^2 + R^2} - R$

$= \sqrt{802.64^2 + 1146.28^2} - 1146.28$

$= \sqrt{644230.9696 + 1314957.8384} - 1146.28$

= 1399.71 - 1146.28 = 253.43 ft. ans.

(b) Mid. Ord. = R (1 - cos I/2)
= 1146.28 (1 - cos 35°)
= 1146.28 (1 - 0.81915)
= 1146.28 x 0.18085 = 207.30 ft. Ans.

(216) Sub-Chords

In general a chord of a curve is understood to be 100 ft. in length, and the foregoing consideration of the simple curve was made on this assumption. When a chord of less than 100 feet is taken as an element of the curve it is called a sub-chord. The sub-chord may be nominally 50 feet, 25 feet, or any fractional part of the 100 ft. chord; but its actual length is slightly more than its nominal length, since in measuring upon the arc of the curve it is evident that two 50-foot chords are not equivalent to the two chords of any arc subtended by a 100 foot chord. Shown in Figure 85.

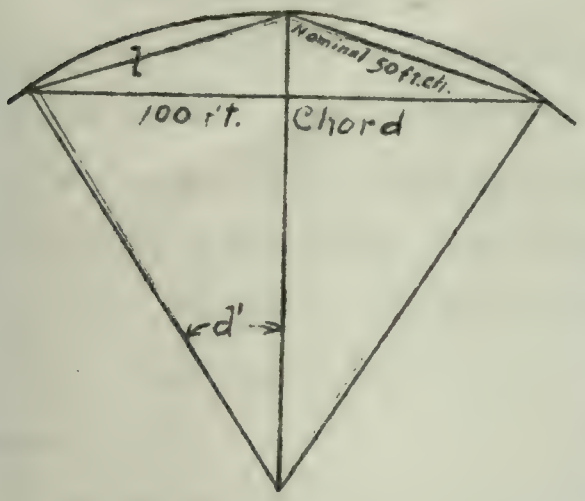


Fig. 85.

The deflection for a sub-chord is one half its subtended arc. Therefore, if l is the length of a sub-chord and d' is the central angle subtended by l, then $l = 2(R \sin d'/2)$. Since $R = \frac{50}{\sin D/2}$ we have by substituting this value for R in the above equation:

$$l = \frac{100}{\sin D/2} \times \sin d'/2; \quad \text{or}$$

$$l = \frac{100 \sin d'/2}{\sin D/2} \quad \text{whence}$$

$$\sin d'/2 = l/100 \sin D/2$$

Since in the case of small arcs the sine is approximately equal to the arc, when the arc D is not greater than 8° or 10°, the error in such assumption (sine = arc) is negligible, and we may write this formula, thus:

$d'/2 = l/100 \times d/2$, or more simply

$d' = l/100\ D$; whence

$l = 100\ d'/D$

by proportion $l : 100 :: d' : D$; or we say the length of the sub-chord is proportional to the arc.

Of course for arcs greater than 10° the error of the foregoing assumption must be reckoned with.

In the figure (fig. 85), it is evident that a correction must be added to a sub-chord of nominal length to give the length necessary for say two 50 foot sub-chords to reach a point on the curve also reached by a chord of 100 feet from the same initial point.

Should it be required to use a 50 foot tape in setting out a curve, then the distance should be determined by formula given above and this length used in the work.

(217) Example:

To lay out a 15° curve with a 50 foot tape, what correction must be added to the tape?

Here $D = 15°$, $D/2 = 7°30'$ and $d'/2 = 1/100 \times D/2$

$d'/2 = 50/100 \times 7° 30' = 3° 45'$

$$l = \frac{100 \times \sin 3° 45'}{\sin 7° 30'} = \frac{100 \times 0.0654}{0.1305} = 50.107 \text{ ft.}$$ Ans.

(The answer is carried to the third decimal place for illustration only.)

When a curve begins with a sub-chord, as in the case where the P.C. does not fall at a full station but at some distance beyond called a plus, for example: P.C. falls at 29 + 35 ft., this nominal length of sub-chord must be increased; thus the nominal length of this sub-chord is then 100 - 35 = 65 ft. If the degree of curve in this

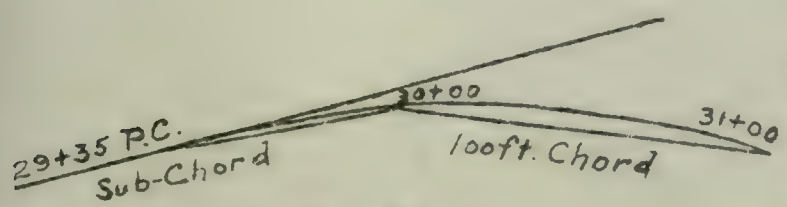


Fig 86.

case is 14°, then D/2 = 7° 00'; d'/2 = 65/100 x 7°00' = 4°55 or 4°33'; therefore, $l = \frac{100 \times \sin 4° 33'}{\sin 7° 00'} = \frac{100 \times 0.0793}{0.1219} = 65.093$. (In setting out this curve the measurement is carried to the hundredth foot only, i.e., l = 65.09 ft.)

The location of curves, their mapping, etc., will be treated in Assignment 30.

Reference is made to Searles & Ives, Field Engineering, pp. 44-60.

Problem to Accompany Assignment 29

Problem:

At a point 375.8 feet from station 237 on a line of road the deflection is 36° 00' R. Find the elements of a six degree curve.

(Find number of chords, length of sub-chords both nominal and corrected length, radius of the curve, tangent distances, the plus to P.C., at P.T., long chord, middle ordinate, and the external distance.)

Problem to Accompany Assignment 29

Reference is made to Searles & Ivas, Field Engineering, 25th Edition.

The location of curves, their weighing, etc., will be treated in Assignment 30.

UNIVERSITY OF CALIFORNIA EXTENSION DIVISION

Correspondence Courses

Surveying-1B Assignment 30 Mr. Swafford

Mapping, Location, Grading. The Planimeter and Its Use.

Foreword

Methods of plotting of points, lines, and details, and also the locating of railroad lines both on paper (the peper location) and in the field are treated in this assignment. A description of the planimeter and its use in measuring areas of maps, diagrams, etc. will also receive attention.

(218) A map is a pictorial representation of a portion of land or water surface. It may range from the simplest outline or sketch to the finished picture, giving details of houses, walls, fences, and other salient features. Generally, in surveying, it is a representation to a given scale of the angles, lines, streams or other bodies of water, ridges, valleys, and slopes which have been secured by surveying methods and are embodied in suitable records called notes of the survey.

In order to make a map it is necessary to have notes or record of a more or less complete character. The completeness of the map will depend upon the amplitude or scope of the data contained in the notes. It has already been made clear that notes should omit no detail in the way of data that can materially assist in producing an effective map. There should be locations and measurements of points,

lines, and angles and their relative positions together with ample descriptions. These are often best shown by naming, numbering, and picturing in proper sketches.

A line is shown in length (to scale) and angle (bearing, deflection, etc.).

An angle is represented in its true magnitude in units of circular arc (known as "angle").

Elevations and depressions, slopes, valleys, and hills are indicated by conventions called hatchings or contours. Streams are shown by the lines that picture irregular water courses as they occur at lowest valley lines or "thalwegs" (a German word meaning "valleyways"). The convention for streams calls for lines that are very fine near the source and that gradually increase in width toward the mouth or point of discharge; in larger streams where the scale permits, two lines representing the banks of such water courses are employed in the representation.

Lines, when straight, are drawn by means of a ruler (straight edge, T-square, triangle); when curved, by use of the compass or a curved ruler; when irregular, as meander lines, profiles, etc. by short straight free-hand lines from point to point as previously determined.

Angles are set off by one of three principal methods; by tangents, by chords, or by protractor. These three methods are illustrated in the following figures and concrete examples, by which angles of bearing, deflection, or included angle are set off or mapped.

(219) Tangent Method

To set off an angle of bearing by the tangent method, measure a distance from the vertex of say ten inches along the meridian; from a table of natural tangents take the tangent of the bearing which should be multiplied by 10. Upon the perpendicular drawn to the meridian of ten inches, lay off this distance (10 x tan. of bearing); through the points thus determined draw the required line which will make the required angle the meridian.

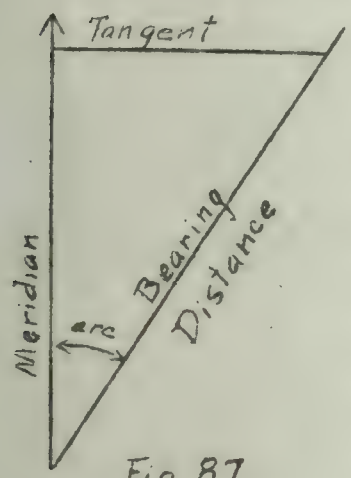


Fig 87

Example: On a scale of 1 inch equal 200 feet draw a line 750 ft. long, having a bearing N 38° 40' E. Here SN represents the meridian and the required line has its origin at A. Lay off 10 in. along SN from A, giving the point m. Erect a perpendicular mn equal to 10 x tangent (natural) of 38° 40' (10 x 0.800 = 8.0). Through An draw the line AB making this 750 ft (by scale 3.75 inches). For convenience, construction lines have been drawn to a scale of 3/16" = 1".

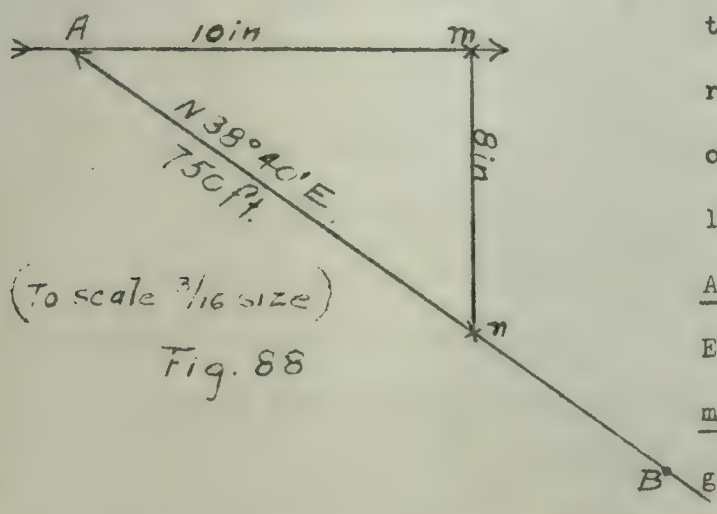
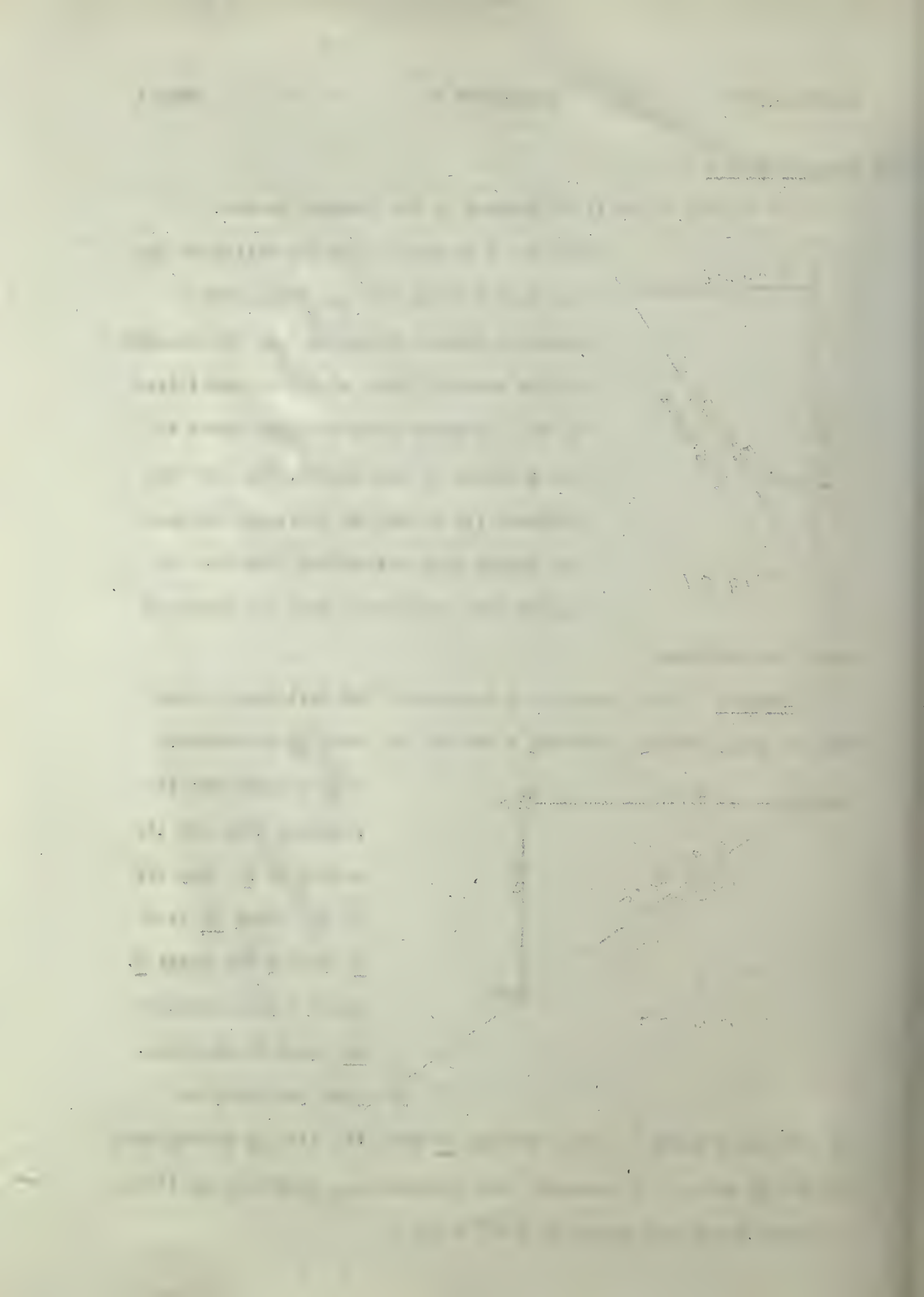


Fig. 88



Since the tangents of angles over 45° are greater than one and the tangent increases rapidly, reaching infinity at 90°, for angles greater than 50° it is better to use the cotangent instead of the tangent. To illustrate: Lay off a line that deflects 78° 41' R by use of the cotangent as shown in Fig. 89. In this case through the vertex A draw AL perpendicular to the directed line AM; lay off 10 in. on AL (= Am); at m erect the perpendicular mn making mn = 10 x cot 78°41' (10 x 0.200 = 2); now draw AB from A through n. The angle MAB is the required angle. Had it been attempted to set up the tangent of 78° 41' at M it would have required a line 10 x 4.99695 inches long or nearly 50 inches, much beyond the limits of the ordinary drafting board.

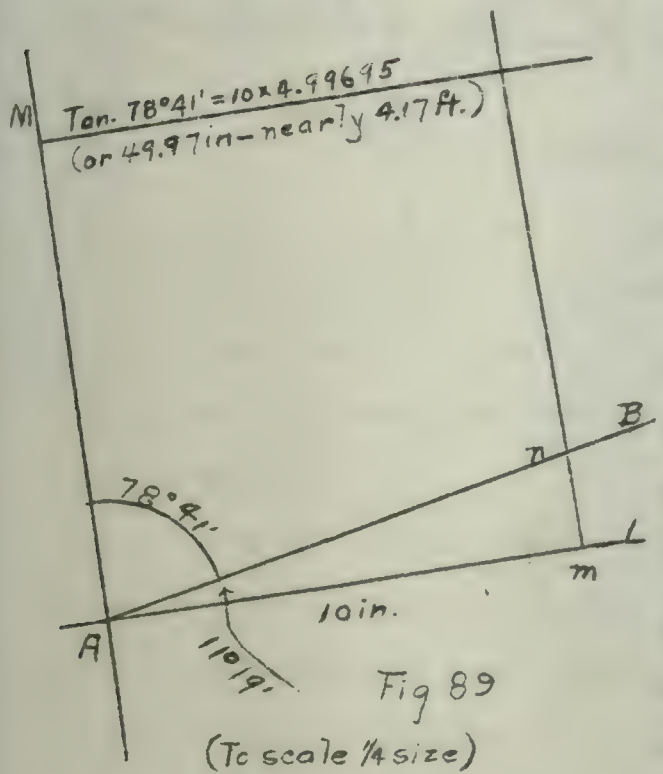


Fig 89
(To scale ¼ size)

The tangent method is especially applicable and convenient in plotting open traverses such as are made in road and railroad surveys. A study of Fig. 90 will sufficiently illustrate this method.

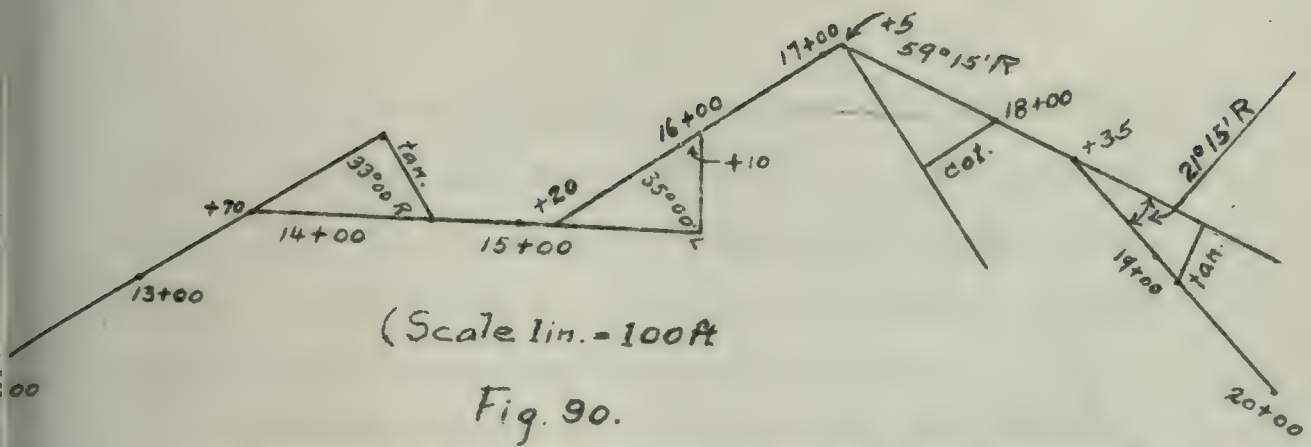


Fig. 90.

In general the angles here are small and of course the tangent is properly used; but at station 17 + 05 the deflection angle being 59° 15' R the cotangent is more conveniently applied.

(220) The Chord Method

The chord of an arc is the line joining the extremities of the arc. Tables are sometimes, but not commonly, included in the tables of engineering handbooks. Referring to Fig. 91 it will be seen that the chord of any arc of unit circle is equal to twice the sine of one-half the angle:

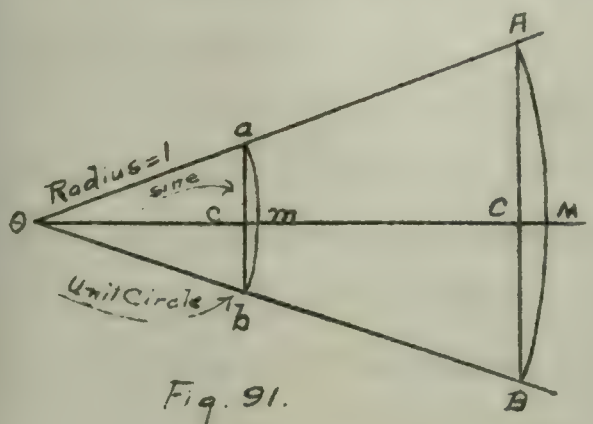


Fig. 91.

In the unit circle Radius = 1; sin angle $\theta = \frac{ac}{\theta a} = \frac{ac}{1}$; angle θab = 2 angle $a\theta c$ and $ac = cb$ or $ab = 2ac$; i.e., chord ab = 2 sin angle $a\theta c$. Now in $A\theta B$, the arc AMB subtends the same angle θ and the chord AB; by similar

Fig 80.

triangles $\Theta a : \Theta A = ab : AB$; but Θa = Radius = 1, and $\underline{ab} = 2 \sin \Theta 2$; $\therefore 1 : \underline{\Theta A} = 2 \sin \text{angle } \Theta/2 : AB$; and $AB = \Theta A \times 2 \sin \text{angle } \Theta/2$.

If in a special application of this method we make the radius of the construction arc 5 inches long, then the chord is equal to $10 \times \sin \Theta/2$. That is, for any given angle, take the natural sine of half this angle, move the decimal point one place to the right and lay this off as the chord to a radius of 5 inches.

In plotting, describe the arc and from the point where it cuts the line of direction scale off the straight line, $10 \times \sin \Theta/2$, with the proper scale; draw the required line through the point thus found.

This method is applicable to plotting of any sort and when tables of chords are available is very satisfactory and simple. It is especially applicable to plotting of interior angles. Where the angle is large, exceeding 70°, it is better to use complementary construction, using sine of half the <u>complement</u> of the required angle.

(221) <u>Protractor Method</u>

The protractor is a universal instrument for plotting and measuring angles of any sort, such as bearings, deflections, or azimuths. The protractor is made of various kinds of materials, as paper, brass, celluloid, silver, etc., and of sizes and styles suited to the various needs of mapping. Exquisite instruments made of silver with accurate and small subdivisions and supplied with verniers, are to be had from the instrument makers, and the map draftsman will

find use for these where great accuracy and many angles are to be laid out. A good plain protractor is usually all that is required, and in work of some kinds a large paper protractor is sufficient and convenient. Such paper protractors may be purchased in several sizes, 8 inches and 14 inches diameter being common sizes. The larger size is divided into 1/4 degree divisions; to set off angles closer than 15 minutes of arc it is possible to estimate smaller units as 7.5 minutes or even 5 minutes. This is usually within the required limits of accuracy in construction but where higher accuracy is desired, the tangent method should be used.

Generally it is convenient to have the protractor in semi-circular form; it is so used that the center of the circle is placed at the vertex of the angle to be plotted with the diameter of the protractor along the meridian or directed line, the angle in degrees and fractions may then be set off by the pencil point or pricker and the line drawn through the points thus determined. A protractor of semi-circular form may be conveniently used against a straight edge or T-square placed along the meridian; the center at the angle vertex.

For plotting azimuths, especially when it is required to locate many points, as on detail or topographic maps, where the location is shown in the notes by azimuth and distance, the full circular protractor is most convenient. For such use, cut out the protractor in the form shown in Fig. 92. This may be done with a sharp knife along fine pencil lines previously drawn in the following manner:

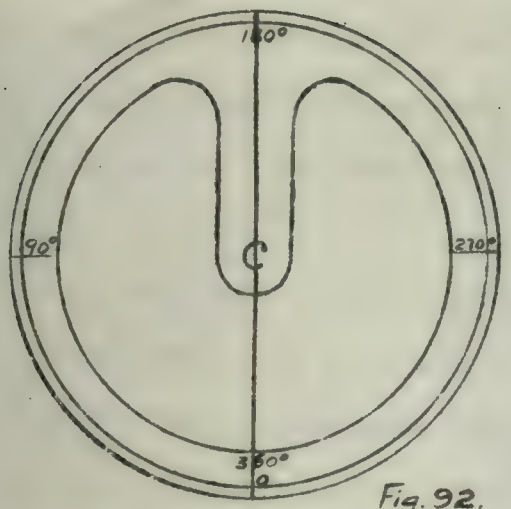


Fig. 92.

About the center (accurately determined by intersections through the quadrantal points), describe a circle just outside (1/16" or 1/8") of the graduated circle. Another circle about 7/8" inside is then drawn, and a tongue about 2" wide, provided to preserve the "center", is shaped as shown; cut out carefully to the lines, removing the U-shaped portion to allow lines to be drawn or seen within the graduated circle.

Now, if azimuths are reckoned from the south, put the center upon the point about which azimuths are measured. Place the zero of the protractor (numbered as in the figure) at the south, and azimuths are then determined by the protractor scale. The tongue may be folded, so that the space within the entire circle is open and unobstructed in drawing lines in that portion of the map. Other ways of cutting out the protractor may be used, but this form is found to be very convenient.

(222) Method of Plotting by Co-ordinates

A map, field, or open traverse is often conveninetly and accurately plotted by means of coordinates. The coordinates may be simple latitudes and departures or the Cartesian coordinates, the latter usually obtained from the latitudes and departures. The

origin of coordinates is in most cases taken at the lower left hand corner of the sheet and so chosen that the most westerly point shall be upon the Y axis, the most southerly point upon the X axis. By this method the ordinate of any point is the total latitude and the abscissa the total departure reckoned from the south and west, respectively.

By means of a T-square and scale the locations of abscissae are rapidly made; the set-square or triangle and scale are similarly used for locating ordinate distances.

Before converting latitudes and departures to coordinates they should be carefully adjusted or balanced for accuracy in plotting. A small error of closure gives a fair indication of the precision of the mapping and this error should fall well within the required limit of accuracy.

The coordinate method of plotting is both accurate and rapid. Since the work of computation of the coordinates themselves is the important part, this should be done carefully, applying the usual checks.

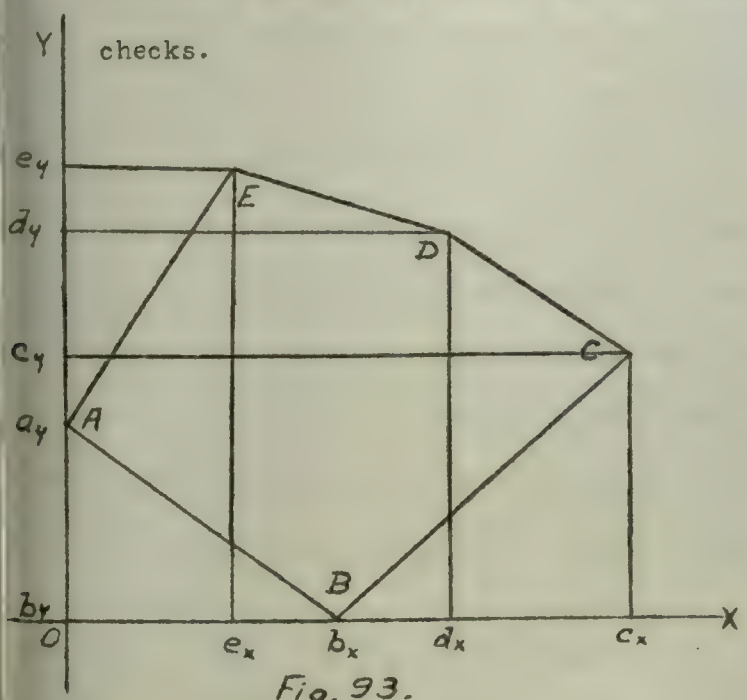


Fig. 93.

The actual plotting is accomplished by means of scale and T-square. Fig. 93 illustrates the method of plotting by coordinates.

Here the axes of coordinates are made to pass through the most southerly and most westerly points of the traverse; hence ay the ordinate of A is found on the Y axis; ax, the abscissa of A is zero; so the abscissa of B is bx measured along the X axis, by falling at the origin. C is located by scaling the ordinate Cy on the Y axis, Cx on the X axis, and by lines perpendicular to the two axes at Cy and Cx respectively. The point C is determined by their intersection. It should be noted that the coordinate axes should be at right angles to each other, and if the T-square is used to draw the lines, as dyD and dxD, the edges of the drawing board should be likewise square. If this condition cannot be secured it is preferred to use a triangle or set-square for drawing one set of lines.

The plotting of details within any traverse or triangulation scheme is best done by angle (bearing or azimuth) and distance.

(223) Final Location of Railway

In mapping a final location of a railroad the tangents are drawn from intersection to intersection and the P.C. and P.T. of any curve are located by tangent distance from the P.I. Arcs of radius equal to the radius of the curve are then drawn from P.C. and P.T. and their intersection becomes the center of the arc of the curve which is then drawn. The intersection angles are best laid off by the tangent method of construction. By reference to Fig.94 this method may be readily understood. The line haveing been carried to P.I., the intersection of angle I is constructed by the tangent method. Then from P.I. the tangent distances to P.C. and P.T. are

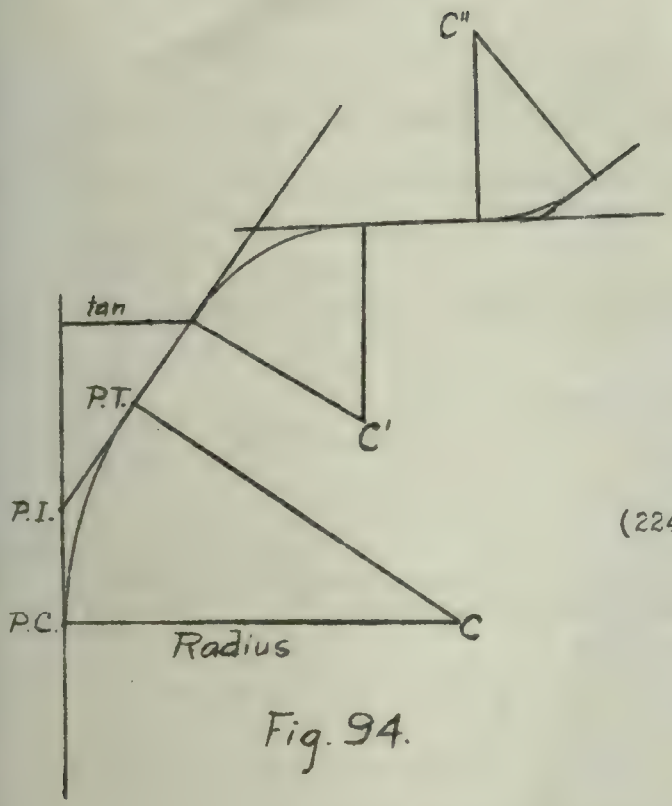


Fig. 94.

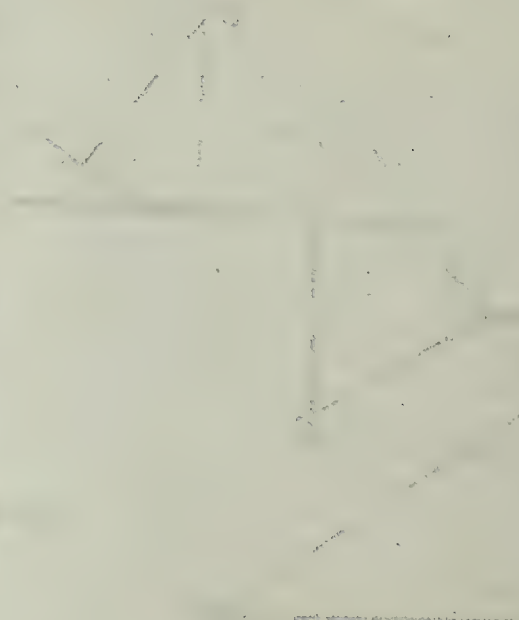
measured off. With P.C. and P.T. as centers describe arcs of radius equal to the degree of curve intersecting at C. Then with the same radius and C as center describe the curve P.C. to P.T.

(224) Areas

In general areas are computed directly from data procured in the survey; as, when a field is surveyed by measuring the triangles composing it, or by bearing and distance from which we pass to the method of latitudes, departures, and double meridian distances, etc. Maps plotted to scale are sometimes drawn on coordinate paper and the area thus measured. But if a map has been carefully drawn to scale the area of the map of any portion may conveniently and rapidly be determined by the means of an instrument called the planimeter.

(225) The Planimeter

In its simple form the planimeter consists of two radial arms hinged together; one arm carries a wheel turning upon an axle, the center line of which is parallel to this arm, which has at its



extremity a tracing point that is made to follow the bounding line of the figure whose area it is desired to measure. The extremity of the other arm has a point, called the anchor point, by means of which this part of the instrument may be fixed to the surface of the diagram.

Figure 95 shows a common form of the planimeter, named from its inventor the Amsler planimeter. K is the arm to which the anchor point A is attached; h is the arm bearing the tracer point, T, These two arms are hinged at J and are so shaped near the hinge that they may be folded together about the wheel W. The axis of W is made parallel to the arm h, and upon the axle is usually a worm gear that operates a disk for counting the revolutions of the wheel. The wheel is subdivided into 10 numbered divisions, each of which is again subdivided into 10 equal parts. A vernier is attached to secure a reading to one-tenth of these smallest divisions. The counting device consists of the disk that gives the whole number of revolutions of the wheel, the numbered divisions on the wheel give the 10th, the smaller subdivisions show the hundredths of revolutions, and the vernier gives the thousandths. Figure 96 shows a reading of 4.516 - 4 is taken from the disk C, 5 from the numbering on the

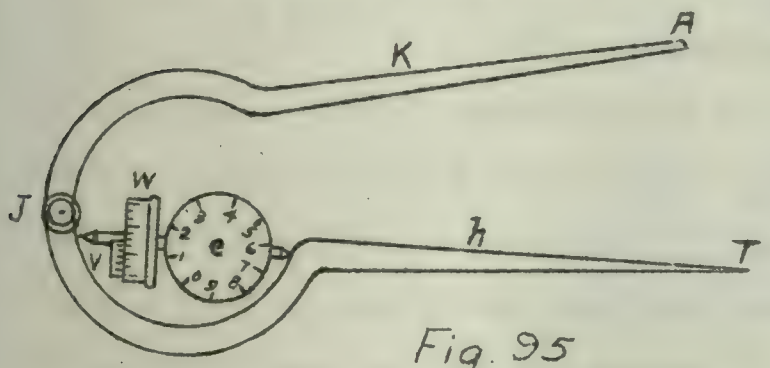


Fig. 95

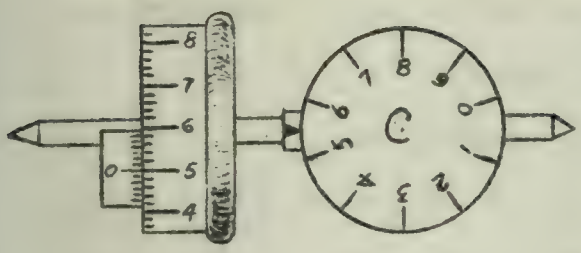
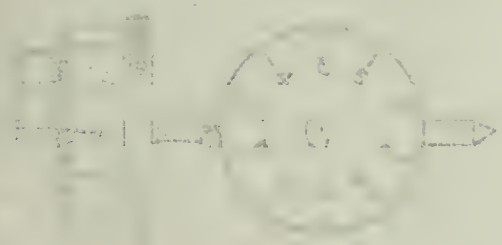


Fig. 96.

wheel, 1 (the vernier zero has passed this point), which with the vernier reading 6, completes the recorded number of revolutions.

The action of the instrument is as follows: The anchor arm is fixed to the diagram and the tracer point carried around the bounding lines of the figure to be measured. The wheel either rolls or slides over the surface; it rolls when the direction of motion is at any angle to the tracer arm greater than zero and slides (without rotation) when the direction of motion of the tracer is in line with the tracer arm. The rolling of the wheel is either direct (i.e. the numbering on the recording device is increasing) or reverse (i.e. the numbers decrease). The arrangement of the parts of the instrument is such that if the tracer point is carried clockwise around the figure when the anchor point is <u>outside</u> of the figure, the numbering is increasing, or the reading is <u>direct</u>; if the tracer point is moved counter-clockwise, the wheel rolls <u>negatively</u>. Therefore, we speak of the action of the wheel as having a <u>positive</u> or <u>negative</u> roll. (<u>Caution</u>: The instrument should be used on a smooth but <u>not</u> a glossy surface. Smooth drawing surface is best.)

Without discussing the theory of the planimeter it will be sufficient to state that the relation of the parts is such that the



area of the figure traced is equal to the product of the tracer arm times the length of the roll of the wheel. Or thus in symbols:

$$A = lnc.$$

in which A = the area in square units, l = length of the tracer arm from hinge to tracer point, c = the circumference of the wheel, and n = the number of revolutions of the wheel. All of these quantities are measured in the same units, i.e. in inches, or centimeters or feet, or other desired units.

Example: The length of a tracer arm is 5 inches, diameter of wheel 0.75 inches; if the number of revolutions is 4.516, what is the area of the figure traced when the anchor point is outside of the figure?

$A = 5 \times 0.75 \times 3.14 \times 4.516 =$ 10.64 sq.in. $(A = l \times d \times \pi \times n)$

When the anchor point is fixed within the area to be measured, the behavior of the wheel is such that the number of revolutions recorded will give, not the area of the figure, but the area of the figure minus a certain area known as the area of the zero circle (or zero circumference). This is better known as the correction area, since this area must be combined with the result obtained when the anchor point is within the bounding line of the figure traced. We shall call this quantity the "correction area".

(226) The Constant of the Planimeter

For any fixed length of arm and circumference (diameter) of wheel, there is evidently a constant, which is the product of these two factors in the formula A = lnc, n being a variable depending

upon the extent of the area traced.

Without actually measuring the length of arm and the diameter of the wheel this constant, lc, may easily be determined as follows:

1. Lay out a figure of known simple dimensions, as a square 5 in. by 5 in., or a circle of convenient radius, the area of which is easily computed, or a rectangle of suitable dimensions; the purpose in any case being to have a figure of suitable size whose area may be readily and precisely computed.

2. Fix the anchor point outside of the figure and in such position that the tracing point may be carried around the bounding lines conveniently. After setting the anchor point trace the lines to see if this condition obtains.

3. Note the initial reading of the counting device when the tracing point has been set at some point on the bounding line. This is best at a corner, if the figure is made up of straight lines, or at a marked point on the circumference if a circle.

4. Carry the tracer around the figure clockwise until it has passed over the entire boundary and read the counting device for the completed record. The difference between the initial and the final readings is the number of revolutions, n, for this measurement.

5. We now have the area of the figure, A, and the number of revolutions, n. Substituting these quantities in the formula $A = lnc$, we may find the constant of the instrument lc; for:

$$lc = A/n.$$

Likewise we may measure the length of the tracer arm (from the hinge to the tracer point) and find c, or the diameter of the wheel may be measured directly and c computed from $c = \pi d$.

But it is sufficient to determine the constant lc, as explained above, and any areas measured are then easily computed by multiplying the number of revolutions (n) in any case by the constant to obtain the area.

In determining the constant as above it is always best to make several trials and take the mean of these as the most nearly correct value.

(227) To find the value of the "correction area" construct a figure of simple dimensions, fix the anchor point within the bounding line and trace the figure noting the roll of the wheel. When the tracer is carried clockwise and the roll is positive we get a positive value for n (i.e. +n) but if the roll of the wheel is negative then we have a negative value for n. In the same way if the tracer is carried counterclockwise and the roll of the wheel is positive we have -n; if the roll of the wheel is negative we have +n. Care must be taken to observe these facts.

The value of the result as thus obtained is $lnc = Ak - Ac$, in which lc is the constant (determined as above), n is the number of revolutions and may be either positive or negative, Ak is the computed area of the known figure, and Ac is the correction area.

Hence the correction area,

$$Ac = Ak - (\pm nlc)$$

It may be observed that if the roll of the wheel is positive the correction area is smaller than the area traced; if the roll of the wheel is negative the correction area is greater than the area traced; also, if the difference between the initial and the final readings is zero, the correction area is equal to the area of the figure traced. Stated thus:

$$Ac = Ak - (lc.+n) = Ak - lcn, \text{ Ak greater than Ac}$$

$$Ac = Ak - (lc.-n) = Ak + lcn, \text{ Ak less than Ac}$$

$$Ac = Ak - (lc.zero) = Ak - 0, \text{ Ak} = Ac$$

In using the planimeter for finding the area of a portion of a map in square miles, acres, etc., it is necessary to take the scale of the map into consideration. If the scale is given so many acres to the square inch, or so many square miles to the square inch, then it is only necessary to reduce the number of square inches to acres (or square miles) by multiplying by the proper ratio. The same would be true where the area is found by planimeter in other units and the value of one unit known in some other unit. To illustrate:

Suppose the map were drawn to scale of 50 miles to the inch, then the square inch would represent 2500 square miles. Again, if the scale were 1 inch = 400 feet, then 1 sq. in. would equal 160,000 square ft., or $\frac{160000}{43560}$ acres. Hence, if the area were found by planimeter to be m sq. in., then the area in acres would be $\frac{160000\ m}{43560}$ acres. (1 acre = 43,560 sq. ft.)

The simpler makes of planimeter have the tracer arm fixed (not adjustable) hence the constant lc of such having been determined for

$\frac{150000}{6000}$

any given unit area, the number of revolutions alone constitutes the only variable to be observed. Other makes have adjustable arms and are provided with a graduated scale or gauge-marks that enable the setting of the instrument to read directly square centimeters, sq. meters, sq. inches, acres, etc.

The planimeter is made in many styles and adapted to many purposes such as finding the areas of cross-sections, profile sections, steam engine indicator diagrams, land surfaces, and in fact finding any area that may enter into a general or specific problem.

References

Breed & Hosmer, Vol. I, pp. 445-461

Raymond, See index and pp. 172-179

Johnson, pp. 262-270, and pp. 143-161

Tracy, pp. 485 to 500

Problems to Assignment 30

1. A quadrangular field has the following:

	Bearing		Defl. angle
A-B	S 30° 00' E	459	
B-C		434	
C-D		521	117° 00' L
D-A		397	

Int. Angles : ABC = 115° 30'; CDA = 109° 30'

Construct the figure by tangents and chords, scale 1 inch = 5 ft. Accompany drawing with complete data.

2. Find the constant (lc) of a planimeter which in tracing three times the perimeter of a square 4 in. x 4 in. gave: initial reading = 2.734, a = 4.328, b= 5.926, c = 7.527.

3. If the tracer arm is 5 inches, what is the diameter of the wheel of the planimeter used in problem 2?

4. The same planimeter was used to trace a portion of a map with scale 1 inch = 4 miles, initial reading 3.472, final reading 5.555.

UNIVERSITY OF CALIFORNIA EXTENSION DIVISION

Correspondence Course

Elements of Surveying

Surveying 1B Swafford

Assignment 31

Triangulation and Base-Line

Foreword:-

This assignment will treat of Triangulation systems as used with Base-Lines in surveying, the methods of measuring a base-line and the location of triangulation stations and the adaptation of the method to topographic surveying.

(228) Base-Line:-

Primarily, any system of triangulation begins with a base-line accurately determined by the most precise methods of measurement and its location also accurately determined by astronomical observation.

The region covered by a triangulation net is first carefully reconnoitred to obtain the most advantageous points for purposes of observation. These are determined by their prominence, intervisibility, and accessibility for the purpose used. The situation of triangulation points should be, generally, a prominent one, such as peaks or summits, headlands, steeples, light-houses, etc.; but where the points are to be used for observing stations, it must be possible to occupy them with a transit or theodolite. Therefore some of the above described points would have to be eliminated from the list given.

Another consideration is that of the location of the proposed base-line, which should be chosen in a comparatively low altitude and preferably in a straight and nearly level stretch of sufficient extent to permit its being projected in a straight line or nearly so. If the region covered by the triangulation system embraces hills, valleys, woods, and plains, it is evident, therefore, that much attention must be given to the work of reconnaissance, so as to overcome possible obstructions to the proper carrying out of the work.

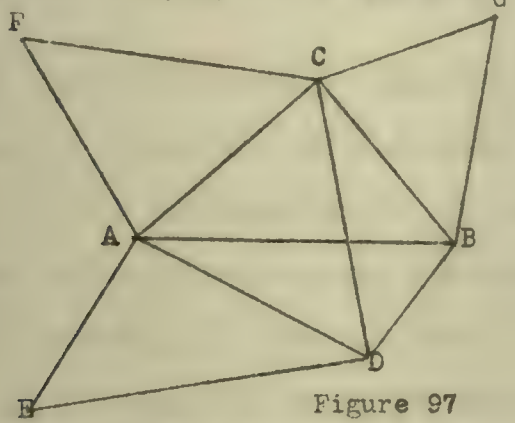
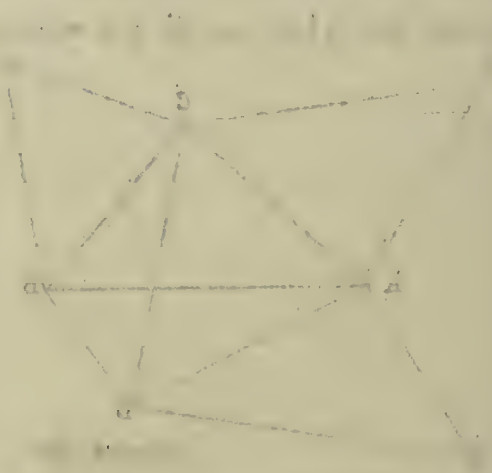


Figure 97

(229) Suppose the Base-line AB in the adjoining Fig. 97, representing what is called a <u>base-line net</u>, has been established and accurately measured as subsequently described in this assignment. The points C, D, E, F, G, etc. are then chosen so that the work of measuring the necessary angles in the configuration shown may proceed unobstructed. It is essential to good work in this case that A, B, C, and D shall be intervisible each from all the others, and that the triangles ABC, ABD, ACD, and BCD shall be well proportioned. By this is meant that the sides should be nearly equal. None of the angles should be less than 30° or more than 120° and the more nearly they approach 60° the better. The diagonal CD, being especially for the



purpose of checking, need not make large angles with the other lines, but it would be best to secure angles greater than 30° if possible.

In the triangle ABC the line AB (base-line) is known; hence, to find the sides AC and BC, the angles CAB and CBA are carefully measured with a transit or theodolite, and the law of sines applied, thus:

$$AC = \frac{AB \cdot \text{Sin } d}{\text{Sin } C} \text{ , and } BC = \frac{AB \cdot \text{Sin } a}{\text{Sin } C}$$

So also in the triangle ABD, the sides AD and BD may be computed from the known side AB (base-line) and the angles at A and B.

The parts AC, BC, AD, and BD having been determined, and the angles included having been measured, we may proceed to compute the diagonal CD. This completes a quadrilateral known as the base net, and since the subsequent expansion of the triangulation system must depend for its correctness upon the accuracy with which the parts of this quadrilateral are determined, it is essential that careful heed be taken to perfect the measurements and computations upon which the determined parts depend.

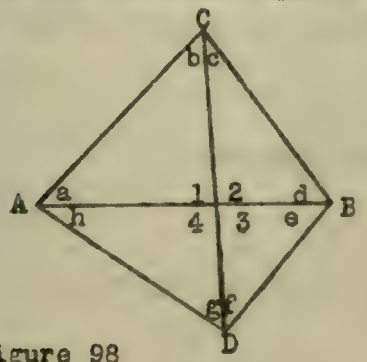


Figure 98

The angles of the quadrilateral are measured either with a theodolite or a transit; if the transit is used the method of measuring angles by repetition is employed. The

transit is set up at A, the instrument carefully centered over the exact end of the base-line and the plates carefully leveled. Too great care cannot be taken to set the vertical axis vertical, and to have the tripod firmly planted and the instrument in perfect adjustment. The accuracy of the results in angle measure will depend much upon the handling of the instrument; especially is this true with respect to the clamping of plates and the use of tangent screws; also attention must be given to the manner of turning the transit on its vertical axis in sighting and, again, in the inverting of the telescope in the act of "plunging". Clamp-screws should be brought firmly to seat, but never clamped very tightly. The very act of urging the screw against its bearing may throw the instrument out of level or cause a slight rotation of the whole about the vertical axis. A tangent screw will act better when opposing the spring, than when the spring follows the screw; so it is better always to set the vernier index before clamping so that a clockwise movement of the tangent screw will be required to bisect the signal or point sighted upon. During the series of readings by repetition the plate of the transit should not be disturbed, even if it appear that the vertical axis is not truly vertical; if it is thought best to readjust the verticality of the instrument, this must be done and a new set of readings made

(230) Repetitions of angle should be made as follows: With

the telescope normal, set the A vernier at zero and check by observing the B vernier. Record both readings. Unclamp the lower motion; set intersection of cross-hairs on left-hand signal (i.e. when instrument is at A, sight on C); bisect precisely by means of the lower tangent screw. (Caution: The signal, at C for example, may present a phase such that one side of the signal rod, plumb-bob or other device shall be mistaken for the middle of the same; endeavor to eliminate such a condition.) Unclamp upper motion and sight right-hand signal on B; clamp the upper motion and perfect the bisection of signal with upper tangent-screw. Read and record full angular readings of both A and B verniers. Continue the repetitions by setting on left-hand signal with lower motion, then upon right-hand signal with upper motion the requisite number of times to produce the number of repetitions desired, but recording only the final readings of A and B vernier.

Now (without disturbing either the setting of the instrument or the vernier index) invert the telescope and take an equal number of readings of angle with telescope "plunged". The mean of the two repetitions is the value of the angle measured. Record this as the mean angle.

Let it be supposed that we have just measured angle a at A in Fig. 98. Proceed in like manner to measure angle h at A; then the total angle a + h, and the exterior angle CAD, thus closing the horizon about A. Now proceed to B and measure in turn angle d, angle e, d + e, and exterior angle CBD. Next occupy C, measuring angles, b, c, b + c, and exterior angle ACB; finally,

occupy D, determining angles, g, f, g+f, and exterior angle BDA. All angles are actually measured and not computed from partial observed data; for example, at the last point occupied above, i.e. at point D, measure g and f, then measure g + f; do not assume that the whole angle is equal to the sum of its parts; so also the exterior angle is measured, not computed from 360 - (g + f). The test of the accuracy of the work of measurement is of course that g + f = int. angle ADB; and g + f + ext. angle BDA = 360°. These are checks used in the field and should be so applied, a serious discrepancy being at once corrected.

(231) Having completed the angle measurement at all four stations of the quadrilateral ACBD, before making computations of the sides and the diagonal CD, it is essential that the angles of the quadrilateral should be carefully adjusted.

The following conditions should obtain:

1. The sum of all the angles about any station should equal 360° (the field test should apply).

2. The sum of all the angles of the quadrilateral should equal 360°; i.e. a + b + c + d + e + f + g + h = 360°.

3. The sum of the angles of any triangle should be 180°; e.g. b + a + h + g = 180°, and c + d + e + f = 180°.

4. Since angle 2 = angle 4 (vertical angles at O), then c + d = h + g; also, since angle 1 = angle 3, a + b = e + f.

In making the adjustments it is assumed that the error in any case may be distributed by equal parts to each angle,

1/3 to each angle of a triangle, 1/4 to each angle of a quadrilateral and 1/2 to each pair of equals.

(232) Finally a test of the sides should be made as follows:

Since $AC = AB \frac{\text{Sin } (b + c)}{\text{Sin } d}$, and $CB = AB \frac{\text{Sin } (b + c)}{\text{Sin } a}$, it follows that:

$$AC : CB = AB \frac{\text{Sin } (b + c)}{\text{Sin } d} : AB \frac{\text{Sin } (b + c)}{\text{Sin } a}$$

Cancelling AB sin (b + c) and applying the test of the primitive proportion $\frac{AC}{CB} = \frac{\text{Sin } d}{\text{Sin } a}$ after computing AC and CB.

A further equation of condition for the quadrilateral should also obtain:

$$\frac{\text{Sin } a}{\text{Sin } b} \times \frac{\text{Sin } c}{\text{Sin } d} \times \frac{\text{Sin } e}{\text{Sin } f} \times \frac{\text{Sin } g}{\text{Sin } h} = 1$$

or expressing the same logarithmically:

$$\log \text{Sin } a + \log \text{Sin } c + \log \text{Sin } e + \log \text{Sin } g - (\log \text{Sin } b + \log \text{Sin } d + \log \text{Sin } f + \log \text{Sin } h) = 0$$

The further adjustment of the angles may be necessary in order to bring about the above conditions.

(233) Measurement of the Base-Line.

The site of the Base-line is first determined, the extremities are staked out, and the measurement of its length then made.

Various methods have been employed from time to time, but that now followed is with a steel tape of known standard and used in the field under suitable conditions of wind and temperature,

of a triangle, AA to each angle of a quadrilater-

al and LAZ to each pair of equals.

Finally a test of the ratios should be made as follows:

Since $AC = AB \frac{\sin \beta}{\sin(\alpha + \beta)}$ and $CB = AB \frac{\sin \alpha}{\sin(\alpha + \beta)}$, it follows

that:

$$\frac{\quad}{\quad}$$

$$\frac{\quad}{CB \quad \sin \beta}$$

A further correction or condition for the quadrilateral

is also obtain:

$$\frac{\sin \alpha}{\quad} \quad \frac{\sin \alpha}{\quad} \quad \frac{\sin \alpha}{\quad} \quad \frac{\sin \alpha}{\quad}$$

which gives the proportion:

$$\log \sin a + \log \sin c + \log \sin e + \log \sin g$$

$$= (\log \sin b + \log \sin d + \log \sin f + \log \sin h) = 0$$

generally at a normal tension for compensating sag.

In Assignments 3 and 4, the discussion of linear measurement was given fully and it will be unnecessary here to repeat these instructions; however, it may be advisable to restate the several corrections in tape measurement as applicable to the case in hand.

(a) The tape should be corrected for erroneous length.

This requires that the tape be compared with a standard and the error carefully determined. Suppose that the tape is 100 feet long (nominally) and the amount that the tape is longer or shorter than this when compared with standard is found. The correction to the measured line is applied for every hundred feet of length in the measured line. Again if a fractional tape length remains - say 37.175 feet or 69.827 feet, then that portion of the tape used in making this fractional measurement should be tested, by standard to determine the correction for this special segment. Usually it is known or assumed that the discrepancy between the used tape and the standard is uniformly distributed throughout the whole length of the tape and the correction is so applied.

Hence, to correct for standard, multiply the measured length of the line by the error in one tape length, divide by the length of the standard (100 feet in this case), and add the correction if the tape is longer than the standard, or subtract if shorter than the standard.

This requires that the tape be compared with a standard
and the error carefully determined. Suppose that the tape is
feet long (nominally) and the amount that the tape is longer
or shorter than this when compared with standard is found. The
proportion to the measured line is applied for every hundred feet
length in the measured line. Again if a fractional tape
length remains - say 37.178 feet or 69.827 feet, then the
the tape used in making this fractional measurement should be
tested, or standard to determine the correction for this amount
found. Usually it is known or assumed that the difference
between the used tape and the standard is uniformly distributed
throughout the length of the tape and the corr

$$L_c = L_m \left(1 \pm \frac{e}{100}\right)$$

L_c = corrected length of line, L_m = measured length of line, e = difference between the standard 100 foot tape and the nominal one hundred foot tape.

(b) Each measured segment of the line should be corrected for temperature.

The tape is standard at 62° Fahr.; therefore measurements taken at other temperatures must be corrected as follows:-

The temperature T of the tape is observed usually at two points, a few feet from each end, the mean of the observed values being taken for the value of T. The coefficient of expansion for the ordinary steel tape is approximately 0.0000065 $\pm$ a (The correction $\pm$ a which is very small may, of course, be ignored).

This coefficient is per foot per degree Fahr.; hence multiply the difference in temperature (temperature of tape minus the temperature of standard) by the length of line times the coefficient which gives the correction to be added if temperature is above standard, to be subtracted if temperature is below standard.

$$L_c = l_m \pm (T_1 - T_o)\ 0.0000065$$

In which l_c = correct length of segment, l_m = measured length of segment, T_1 is the temperature (observed mean), T_o is the standard temperature, and 0.0000065 the coefficient of linear expansion.

$$L_c = L_t' \left(1 + \frac{\quad}{\quad}\right)$$

L_c = corrected length of line, L_m = measured

e = difference between the standard 100 foot tape and the

reading one measured object tape.

(3) Each measured segment of the line should be corr-

the true is at about at 62° Fahr.: therefore

a thermal other temperatures must be made according follow-

The temperature T of the tape is measured usually at two

a few feet from each end the mean of the observed values

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ordinary steel tape is approximately 0.0000065 I a (the corr-

+ a which is very small may of course be ignored.

the difference in temperature (temperature of tape when

temperature of standard) by the length of line time the coeffici-

$$T_c = T_m - (T_t - T_s)$$

Where T_c = corrected length of segment. T_m = measured

length of segment, T_t is the temperature (observed mean), T_s is the

, and 0.0000065 the coefficient of linear

(c) Each measured segment should be corrected for slope.

It is evident that even upon a comparatively level stretch, unless the end supports of the tape can be brought to the same elevation, the length of segment, l, is not the horizontal distance but the slope distance for that segment. Hence, a reduction to the horizontal is required.

This may be secured in one of two ways; either the angle of slope must be measured, or the difference of elevation of the two ends of the segment must be found and the appropriate correction computed and applied. The measurement of angles of slope, except with special base-line apparatus (notably the Holden Clinometers, and other instruments) is troublesome; and even with such special apparatus gives no more reliable results than the method here described, as follows:

With engineer's level or transit used as such take rod readings at both ends of the segment; preferably set up instrument equidistant from the two ends thus eliminating error in adjustment of the line of sight perpendicular to the vertical axis. The difference of the two readings so obtained is the required difference in elevation; this difference may be easily found to the nearest 1/100 of a foot which is well within the degree of accuracy desired, as a difference of 1/100 of a foot is about that of 20" of arc, for a distance of 100 feet (one segment of 100 feet).

The tape when compared with standard is supported

(c) Each measured segment should be corrected for slope.

throughout its length, but in the field this is impracticable; therefore, it is supported only at the ends and a correction for sag must be made, or, what is more feasible, the sag may be compensated by applying a tension just sufficient to accomplish this.

(d) The correction for sag is found from the formula: $\frac{L}{24}\left(\frac{wl}{t}\right) = C_s$, in which C_s is subtractive as the length of the segment, is less by the amount of this correction than the actual length. L is the nominal length of the segment (100 ft), l is the length of the same (if supported at the ends, this is also 100ft), w is the weight of the tape per linear foot, and t the tension applied.

(e) The correction for tension or pull is expressed by the formula: $C_p = \frac{Lt}{SE}$, in which C_p is additive since the effect of pull is to stretch the tape, and hence the measured length of the base-line segment is greater than the actual length. Here L is the nominal length of the tape (100 ft), t is the pull or tension, S is the cross-sectional area of the tape in the same unit as L (i.e. the foot) and E is the modulus of elasticity of the material of the tape. The modulus is taken as 30,000,000 lbs., being that of steel used in tapes of this kind.

(f) Normal tension is a tension which applied to the tape compensates for the sag; or since the C_s is a subtractive correction and C_p is additive we may put $C_s = C_p$ and find the

resultant value of t in the equation, thus:

$\frac{L}{24}\left(\frac{wl}{t}\right)^2 = \frac{Lt}{SE}$; and this solved for t gives $t = \sqrt[3]{\frac{SE w^2 l^2}{24}}$

This tension applied to all measurements removes the troublesome computations for sag and tension; the results obtained are equally reliable with other more complex method.

(234) Broken Base-Lines

If feasible a base-line should be measured in a straight line, but as it is sometimes more important that the base should have the desired extent and location of its extremities than that its measured parts should be in the same straight line, a "broken base" is often chosen intentionally, and the broken base is then reduced to a straight base, as follows:

Suppose that the segments AC and CB of a base have been

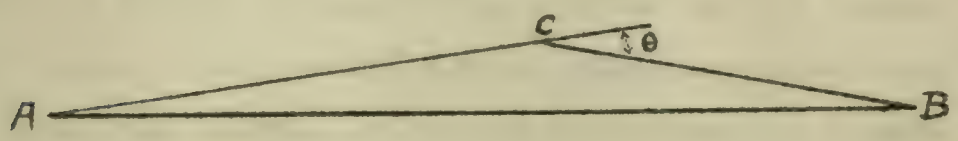


Fig 99.

measured and that these deflect at C by the angle θ. Then the base-line AB is computed from the trigonometric law of cosines, thus:

$AB^2 = AC^2 + CB^2 - 2(AC \times CB) \cos ACB$; or choosing instead the deflection angle θ (180° - ACB) and extracting the square-root of both numbers

$$AB = \sqrt{AC^2 + CB^2 + 2(AC \times CB) \cos\theta}$$

From the above an approximate formula is derived which gives satisfactory results when applied in cases where the angle θ does not exceed from 3° to 5°. The formula is:

$$AB = AC + CB + 0.00000004231 \frac{AC \times CB\ \theta^2}{AC \times CB}$$

As logarithmic computation is here desirable the logarithm of 0.00000004231 is 2.626424 - 10.

If the angle θ is larger than 5°, then it would be necessary to measure the angles at A and B and compute AB from the law of sines.

The procedure in the work of measurement is as follows:

1) Stakes are set in line, by transit set up over A, at intervals of a tape length (100 feet). These stakes should be of substantial size (about 2" x 4" - 3 feet long) driven firmly into the ground and cut off square on top. On the tops are fastened small strips of zinc (also 2" x 4") on which lines may be inscribed to mark tape lengths. In lieu of these stakes low tripods may be used which carry suitable block heads on which the zinc plates may be fastened; two such tripods are needed, the rear one being advanced to a forward position for each segment of a tape length, while the other is left in position undisturbed.

2) The measurement is to be made between tack centered hubs at the base-line extremities and the point on the ground must be transferred to the top of the 2" x 4" stake or to the tripod head.

$$\overline{AB} = \sin\theta + \cos\theta + 0.000000000000 \quad \overline{\sin\theta \times \cos\theta}$$

$$\overline{AB} = \sin\theta + \cos\theta + 0.000000000000 \quad \overline{\sin\theta \times \cos\theta}$$

To do this set up a transit at A, carefully leveling same and centering on the point on hub. Turn off an angle of 90° from the base-line and set a tack centered hub 8 to 12 feet distant from A. Invert the telescope and repeat the setting of the off-set point; if the two points coincide, then the off-set point has been properly placed; if the two points do not coincide, take the mean position. Now set up the transit over this off-set point and sight upon the tack-center in hub A and transfer this to the top of the 2" x 4" stake or tripod, marking the point upon the zinc plate. It is necessary to set the point with telescope normal and also plunged, in order to eliminate any lack of adjustment of the line of sight; if the two pointings differ, take the mean position.

3) Having thus marked upon the zinc plate the initial point of the base-line, stretch the tape over this mark and the zinc plate next in order by means of stretcher bars, one at each end, with dynamometers (spring balances) attached for determining the pull. If a normal pull is used, and it is better to use a normal pull, see that the dynamometer records this tension at the instant of comparing, or setting off the distance. A line should be drawn on the zinc plate in the direction of the line at each station and a short line at right-angles to this over the mark, on the tape chosen for the limit of the measurement; it is best to make this limit the 100 foot mark or 99.9, 99.8 or some other definite division. Of course for computation, the 100 foot length is simplest.

sion. Of course for construction, the 100 foot limit is simpler
limit the 100 foot rank or 99.9, 99.8 or some other definite dif-
tween the limit of the measurement: it is best to make t

4) While the tape is still in the position used in measuring (i.e. while still suspended in air and thus free from the warmer or colder earth, place a thermometer upon the tape first at one end and then at the other, and record the mean of the two readings, as the temperature of the tape.

5) With a transit or engineers level, set up equidistant from the two ends of the segment, take rod-readings at both ends, rod held on top of the stake or tripod. The difference of these two rod readings will be the difference in elevation between these points.

Proceed in like manner with the second and each succeeding segment, being careful to keep the tape off the ground, and flat with draduations uppermost so they may easily be read, and to observe temperature, tension, and elevation in each case.

6) Record all data as observed in a neat tabular form; the following is suggested:

Seg.	Meas. Length	Mean Temp.	Elevation a	Elevation b	Elevation diff.	Corrections Temp.	Corrections Slope	Length Corrected	Total Meas. Length
1	99.9	78°	7.8	10.7	2.9	+0.010	-0.042	99.868	
2	100.0	83°	6.7	8.5	1.8	+0.014	-0.016	99.998	*

* Add the several segments

7) The total measured length of the line should now be corrected for standard, the length of tape being $100 \pm d$, and d is the difference between the tape used (nominally 100 feet) and 100 feet of standard.

The mean elevation of the ends of the base-line above sea-level should now be determined by running a line of differential levels, from some bench-mark.

8) The reduction to sea-level is made by applying the formula:

$$\text{Correction} = \frac{Bh}{R}$$

in which B = the measured length of the base-line, h = the altitude above sea-level and R = the mean radius of the earth.

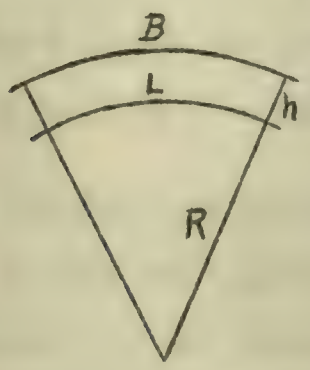


Fig 100

In Fig. 100 let B = the measured base, h = the altitude above sea-level and R = the earth's mean radius; then

B : L :: R + H : R

arcs are proportional to their radii.

$$\frac{B - L}{B} = \frac{R + h - R}{R + h} \qquad \therefore\ B - L = \frac{Bh}{R + h}$$

and since h in the denominator is very small compared with R, it may be ignored.

In conclusion, it is best to take three or more measurements of the base-line; as from A to B, from B to A, and again from A to B. The several tape measurements of each segment should be carefully checked, and the several points be properly aligned. Two dynamometers are better than one, placing one at each stretcher; if one is more delicate than the other, there is some advantage in

putting the less sensitive at the rear end and the more sensitive at the forward end, as it is only necessary to note the approximate pull at the rear end, keeping the zero of the tape exactly on the marks.

Six men can conveniently be employed in base-line measurements, two to attend the stretchers; one at each end to observe and mark the tape lengths; one to observe temperature; and the sixth man to handle the transit for giving line, and making off-sets, reading elevations, etc. The man who reads temperatures should act as recorder as also the transit man likewise and they should carefully check all data on entering same.

Problem to Accompany Assignment 31.

The data for segments 1 and 2 are given on page 15 of this assignment; for the subsequent segments 3 to 8 inclusive the data are as follows:

3	99.8	82°	a) 6.8	b) 9.3
4	99.9	80°	7.3	8.4
5	100.0	79°	7.6	7.4
6	100.0	78°	7.2	6.3
7	99.9	78°	4.8	2.4
8	87.6	76°	5.7	4.3

Compute the measured length of each segment; the total measured length of base-line. Apply the correction for standard: Tape compared with standard was 99.987 ft. long. The line measured was broken base - first part from station A to 4, where it deflected 2° 52', the second part station 4 to end of line.

The elevation of end A above sea-level was 125.8 feet; find the mean difference in elevation, and reduce to length at sea-level.

The mean value of the earth's radius in feet is taken to be 20,890,600.

Log. R = 7.31995

UNIVERSITY OF CALIFORNIA EXTENSION DIVISION

Correspondence Course

Surveying 1B. Elements of Surveying Swafford

Assignment 32.

Topographic Surveying.

Foreword.

A general consideration of the purposes and methods of topographical surveying is the subject matter of this assignment; details of the subject will be treated in the next assignment.

(235) Topographical Surveying consists in the obtaining of essential data of territorial extent and relief for the purpose of depicting on proper maps the forms and elevations, descriptive details, and pictorial representations of the physical features of land and water areas.

To do this many conventional devices, signs, and symbols are employed; these conventions have by long use become more or less fixed in character, but it is often desirable or expedient to reproduce the symbols in the margin of the map when finished. These constitute what is known as the legend of the map.

As the methods employed include both territorial extent and relief forms it is necessary to have resort to the various means for securing both horizontal and vertical control of these representations. The horizontal control is secured by triangulation or by traverse; the vertical control by adopting a suitable datum plane to which, through proper bench marks, the elevations may be referred.

The purpose and extent of the map or its survey must determine the nature of these controls: A triangulation system of control, for example, may be that of some portion of a geodetic or geological system, or it may consist of a special triangulation system built upon a baseline, assumed and properly located, and suitable to the purpose in hand. A traverse may be run enclosing the territory covered in the survey, or if the region is of great extent several connected traverses may be advantageously employed. A closed traverse is generally adopted, as this permits the usual checks in closure both of the courses and the elevations, the latter being carried forward with the angles and distances as the traverse survey proceeds. An open traverse may be used to advantage in some cases; especially is this desirable in surveys for roads, railroads, or canals, the usual open traverse forming the framework of the topographical features.

As over a large extent of territory the principal features would appear as the mountain ranges, the valleys between, and the river courses flowing along the valley ways, so in the smaller areas the ridges and thalwegs and the stream lines constitute the most salient forms.

Neglecting, then, the slighter sinuosities of the streams of any region, the rivers, creeks, and brooks, furnish at once an outline of the reliefs of that locality. The streams flow in the direction of the valleys and at the lowest levels, while the ridges form the heights along water-partings at the crests of watersheds.

Hence, whether the map desired is of large or small extent, it is important first to determine the water-courses of the region; next, principal elevations of prominent peaks and especially, elevations along the intervening ridges which call for location and measurement. These may be considered the skeleton of the map upon which the many minor features and smallest details may be joined.

(236) Instruments used in Topographical Surveying.

For this branch of surveying a larger assortment of instruments is available and useful than for any other branch. An enumeration of these and their application may profitably be noted here.

In the primary triangulation work a transit of refined qualities or a direction theodolite is used for measuring angles. The usual base-line tapes with the necessary accompanying instruments, such as stretchers, dynamometers, and thermometers are employed on the linear measurements of base-lines, traverse lines, etc. Level-rods, stadia rods, and tape-rods are required for the various functions of such instruments. Levels of the several varieties - hand-level, dumpy, Wye, and the high grade level known as the "precise", are used as occasion requires; while the transit with level on telescope is often made to perform the functions of the more specialized engineer's level.

For measuring differences of elevation and especially for the determination of the elevation of chief or critical points

the barometer, either the cistern type or the aneroid, is often suitably employed. The aneroid especially is readily available on account of its portability and convenience.

The transit and stadia and the stadia in connection with the plane-table are valuable as affording a ready means of securing heights and distances; and especially on work of details and in taking topography of minor areas these useful means are much and successfully used.

For reconnaissance topographical surveys the pocket compass and the clinometer are useful instruments, and on account of their lightness and portability are especially in favor for such work. The Brunton Pocket Transit, used much in mine surveying is here favorably mentioned as a comprehensive substitute for compass, level, and clinometer; it may be used as a hand instrument and in the better types is supplied with a "ball and socket" attachment fitting it for use with a Jacob-staff.

The odometer or the more modern cyclometer attached to automobiles is in use where distances traveled upon roads and highways by vehicle are desired. Also the pedometer or a "pace-tally" is convenient in keeping count in pacing work, which is much used in taking topography.

The plane-table calls for special mention, as, in its various grades from the most complete to the simplest form known as the traverse plane-table, it offers a most ready and convenient means of traversing, intersection, radiation, and resection methods of

mapping directly in the field; in solution of the three-point problem (an application of the method of resection) the plane-table has no superior for rapid and efficient work. The description and use of the plane-table in surveying will be discussed in Assignments 34 and 35.

Thus it is seen that use for a large range of instruments is found in topographic surveying in its various stages; and an acquaintance with these instruments and their uses constitutes a large part of the topographers duties.

(237) Transit lines when measured by tape are measured either by leveling the tape or, where the slope is sensibly uniform, the slope angle may be taken and the horizontal distance computed by use of the formula

$$H.\ D. = L \cdot \cos \alpha$$

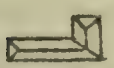
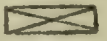
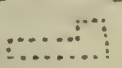
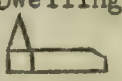
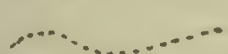
where H.D. is the horizontal distance, L = the measured line on slope and α = the angle of elevation (or depression). Or resort may be had to the formula

$$H.\ D. = L\ (1 - \text{vers}\ \alpha)$$

as the versed sine of small angles is usually a number of few figures and the slide-rule may be used to advantage. For example, a line measured on slope 198.70 ft.; the angle of slope α was 3° 52' (the cosine of which is 0.99772, and hence the versed sine = 1 - cos = 0.00228); by slide-rule 198.7 x 0.00228 = 0.44; then 198.7-0.44 = 198.26

Angles may be taken by deflection, bearing, or azimuth, the azimuth method being generally preferred. Often an angle may be taken easily by reading an interior angle when the adjacent points on both forward and back sights are conveniently marked. If the azimuth record is required the reduction should be made at once and checked in the field.

(238) It is usual to mark transit points on map by a small circle about $\frac{1}{20}$ inch radius enclosing the point, a triangle indicates a triangulation station and a square is used to mark a stadia station; thus ⊙ transit point, △ triangulation station, □ stadia station. The transit point may become either a triangulation station or a stadia station, in which case one sign may be superimposed upon the other as ⊙ or ⊡.

Other conventional topographic signs are: Dwelling House  ; Barn  ; Ruins  ; Church  ; Fence - - - - -, Public Road ; Railroad |-|-|--|-|-| (for small scale); Railroad |=|=|=|=| (large scale); Path or Trail  ; etc. Besides these there are conventions for cover of various sorts, such as meadows or grassy plains, pine forests, deciduous forests, sand dunes, rocky formations, cultivated fields (showing crops of corn, small grain, beans, etc.), swamp lands, and groves and orchards.

All these are usually included in a marginal list designated as "legend" and a map where these or other arbitrary conven-

as may be taken by deflection, bearing, or altitude.
the method being generally preferred. Often an angle may
e taken easily by reading an inferior angle when the adjacent
ints on both forward and back sights are conveniently marked.
Each record is required the reduction should be made at
once and checked in the field.
It is usual to mark transit points on map by a small
pale mount [illegible] each radius bearing the point, a triangle indi-
ted a triangulation station and a square is used to mark a stadi-
tion. Use ⊙ turning point △ triangulation station
— stadia station. The result will show as either a triang-
tion station or a stadia station to which page and line may be
△ [illegible]
Other conventional topographic signs are: [illegible]
[illegible] , [illegible] , [illegible] ,
[illegible] ; [illegible] ; [illegible] ;
[illegible] ; [illegible] ; [illegible] ;
[illegible] ; [illegible] ; [illegible] ;

tions are employed is not complete without a legend.

(239) Since a topographic map is the delineation of the natural and artificial features of any locality upon a plane surface by means of the foregoing conventional signs, it follows that a correct representation should be true to facts by a corresponding faithful use of proper conventions. Every point of the map corresponds to a definite determined geographic position in accordance with some definite method adopted for showing the speroid surface of the earth on a plane; this method is called the "projection". The representation being in miniature (usually a very diminutive scale), the distance between any two points on the map is a certain proportional fraction of the distance between the relative points in nature. This ratio constitutes the "scale".

The points, besides being represented in projection upon a horizontal plane, have their elevations relative to a level surface indicated in some conventional way; the usual conventions are contour lines, depicting points of equal elevation at regular horizontal elevations, or hachures consisting of hatched lines of varying depth of shade or interval.

The level surface to which the elevations are referred is called the datum plane, and this with its system of determined elevations or bench marks constitutes the vertical control of the survey. The representation of the variations in the vertical element with reference to the datum plane is called the "relief", and should fairly represent the modeling of the country.

light depth of shade or tertiary.

The short surface to which the elevations are limited

ting the solid form, and that with the system of determined

tions to other twelve characteristics the addition of the

ing". The interpretation of the notations in the article sta-

ard to the datum line is called the "plan", and

Since all topographic surveys are based upon a system of triangulation or a carefully prepared traverse, a sufficient number of points, whose geographical positions have been determined by either one or the other, or both of the above methods, from the frame work for controlling the less accurate location of the many details. These points should be properly distributed over the area covered in the survey, and constitute the horizontal control.

The determination of these three elements, the scale, the vertical control, and the horizontal control, is fundamental and no survey or its map is complete without it.

As before stated, the purpose of a topographic survey is primarily the securing of the necessary data for the real purpose, the finished map. Hence the observations made in the field should be full and exact enough to secure this final purpose. It is futile to exceed in number or accuracy of data the requirement of the map, but the collection of data must be carried on in an intelligent manner, guided by a more or less complete knowledge of the related sciences and arts employed.

(240) First of all the topographer must be a man of observing habits, endowed with an imagination that will enable him to visualize the relief features of the country he attempts to picture. He must be skilled in use of the pencil and trained to sketch the various relief forms directly in the field as well as to supply some lines and details from numerical data and descriptive notes, in

Since all topographic surveys are based upon a system of triangulation or a carefully prepared traverse, a sufficient number of points, whose geographical positions have been determined by either one or the other, or both of the above methods. From the same work for controlling the less accurate location of the remaining details. These points should be properly distributed over the area covered in the survey, and constitute the horizontal control.

The determination of these three elements, the vertical control, and the horizontal control, is fundamental, and no survey or its map is complete without it.

As before stated the purpose of a topographic survey is primarily the securing of the necessary data for the most map the finished map. Hence the observations made in the field must be full and exact enough to secure this final purpose. It

addition to his field sketches. Moreover, a topographer should know from a study of physical features the causes that have brought about the present outlines of relief which he may be called upon to delineate. He should have a knowledge of the forces which have caused the vast and the recent geological changes, such as the uplifts and subsidences, the volcanic and erosional forces, the action of heat and frost, the wearing caused by ice floes and water; and again the changes effected by erosion or corroding, by the transporting power of streams, and the action of tides and currents on the production of land forms. Not all persons can make good topographers; indeed but few of the many ever qualify for good work beyond the crudest stages of skill and knowledge - for this work is artistic in its nature and hence exacts from him who would successfully follow it, much patience and intelligent practical experience.

QUESTIONS TO ACCOMPANY ASSIGNMENT 32.

1. After reading Assignment 23, state the several characteristics of contours; also get from a dictionary the meaning, spelling and correct pronunciation of the word "contour".

2. Define scale, horizontal control, vertical control, datum, thalweg.

3. What, in your estimation, are the essential qualifications of a good topographer? What are some other desirable qualifications?

References:

Breed & Hosmer	Vol. I. pp. 300 - 320.
Johnson	Chap. VIII.
Raymond	Chap. IX.
Tracy	Chap. XXVII.

UNIVERSITY OF CALIFORNIA EXTENSION DIVISION

Correspondence Courses

Surveying-1B Elements of Surveying Mr. Swafford

Assignment 33

Solution of Problems

Computations and Results

To be returned to student after submitting Assignment 33.

The actual computations of H.D. and V.H. for course A - B are here given. In all other courses and for Side Shots the results only are shown.

Course	H.D.	V.H.	Elevations (Adj)
A--B	385.3	+47.76 (47.79)	B 247.79
B--C	395.9	-26.11 (26.07)	C 221.72
C--D	487.6	+38.66 (38.69)	D 260.41
D--E	630.3	-56.99 (56.95)	E 203.46
E--A	698.4	- 3.45 (3.46)	A 200.00
		+86.42 -86.55	
	Diff	= 0.13	

Computations

$\log \frac{F}{i}$ (98.5) = 1.99344

" S (3.96) = 0.59770

" cos α (7°4') = 9.99669

" " " (7°4') = 9.99669

384.2 = 2.58452

1.1

385.3 = H.D. A--B

- * - * -

$\log \frac{F}{i}$ (98.5) = 1.99344

" S (3.96) = 0.59770

" cos α (7°4') = 9.99669

" sin α (7°4') = 9.08999

47.62 = 1.67782

0.14

47.76 Elev. of B above A

200.00

247.76 Elev. of B above Datum.

Computation of Coordinates for Plotting

Course	Bearing	Dist	Computed Lat.	Computed Dep.	Balanced Lat.	Balanced Dep.	Coordinates Y's	Coordinates X's
A--B	N40°10'E	385.3 ft	+294.4	+248.5	+294.4	+284.3	A 581.6	A 120.0
B--C	N 87°00'E	498.1 ft	+ 26.7	+497.4	+ 26.7	+497.0	B 876.0	B 368.3
C--D	S 25°02'E	487.6 ft	-441.8	+206.3	-441.8	+206.2	C 902.7	C 865.3
D--E	S 50°30'W	630.3 ft	-400.9	-486.4	-400.9	-486.8	D 460.9	D1071.5
E--A	N 41°40'W	698.4 ft	+521.7	-464.3	+521.6	-464.7	E 60.0	E 584.7

Lat Diff = 0.1

Dep Diff = 1.5

For Plotting assume origin of Coordinates at the lower left-hand corner of border on Map. 120 ft. to left of A, 60 ft below E.

Computations of Latitudes and Departures

	A-B	B-C	C-D	D-E	E-A
Lat.	294.4	26.7	441.8	400.9	521.7
log. Lat.	2.46899	1.41612	2.64522	2.60306	2.71744
" Cos.	9.88319	8.71880	9.95716	9.80351	9.87334
" Dist.	2.58580	2.69732	2.68806	2.79955	2.84410
" Sin	9.80957	9.99940	9.62649	9.88741	9.82269
" Dep.	2.39537	2.69672	2.31455	2.68696	2.66679
Dep.	248.5	497.4	206.3	486.4	464.3
	A-B	B-C	C-D	D-E	E-A

H.D., V.H., and Elevation above Datum of Side Shots

Transit at A	H.D.	V.H.	Elev.	Elev. of A = 200.0
1	383	-16.7	183.3	
2	480	-12.5	187.5	
3	426	- 4.8	195.2	
4	270	+ 3.5	203.5	
5	203	0.0	200.0	
6	164	- 4.8	195.2	
Transit at B				Elev. of B = 247.8
7	217	-42.3	205.5	
8	242	-47.6	200.2	
9	377	-54.0	193.8	
Transit at C				Elev. of C = 221.7
10	211	-11.1	210.6	
11	250	-19.7	202.0	
12	232	-24.0	197.7	
Transit at D				Elev. of D = 260.4
13	426	-70.6	189.8	
14	295	-66.0	194.4	
15	275	-65.1	194.3	
16	176	-58.5	201.9	
17	209	-56.2	204.2	
18	202	-50.3	210.1	
Transit at E				Elev. of E = 203.5
19	265	- 4.6	198.9	
20	399	+ 9.8	213.3	

UNIVERSITY OF CALIFORNIA EXTENSION DIVISION

Correspondence Courses

Surveying 1B. Assignment 34. Mr. Swafford.

The Plane-table and Its Uses

Foreword:

Several references have been made to the plane-table in the preceding assignments, and in Assignment 14, page 11, the adjustments of the plane-table were considered; in the present assignment we shall deal with the plane-table and its uses somewhat in detail.

(242) Adaptability of the Plane-table.

The plane-table is adapted to many uses in surveying and is especially convenient in farm or landscape surveys, in topographical work, and preliminary or reconnaissance sketches. In filling in details in triangulation surveys and sketching for relief forms it is particularly useful; the work of the military topographer is usually confined to brief, rapid plane-table sketching.

The plane-table consists of a draughting board varying in size from 12" x 15" to 27" x 36" or larger, altho the larger sizes are more or less unwieldy and hence are not much in favor. A size about 22" x 28" and made of light, but stiff material is most convenient.

The board is mounted upon a tripod by means of foot screws, after the manner of the transit, or by a modified ball and socket arrangement popularly known as the Johnson head, from its inventor.

UNIVERSITY OF CALIFORNIA EXTENSION DIVISION

CORRESPONDENCE COURSES IN ENGINEERING SUBJECTS

PLANE SURVEYING

COURSE X–1B

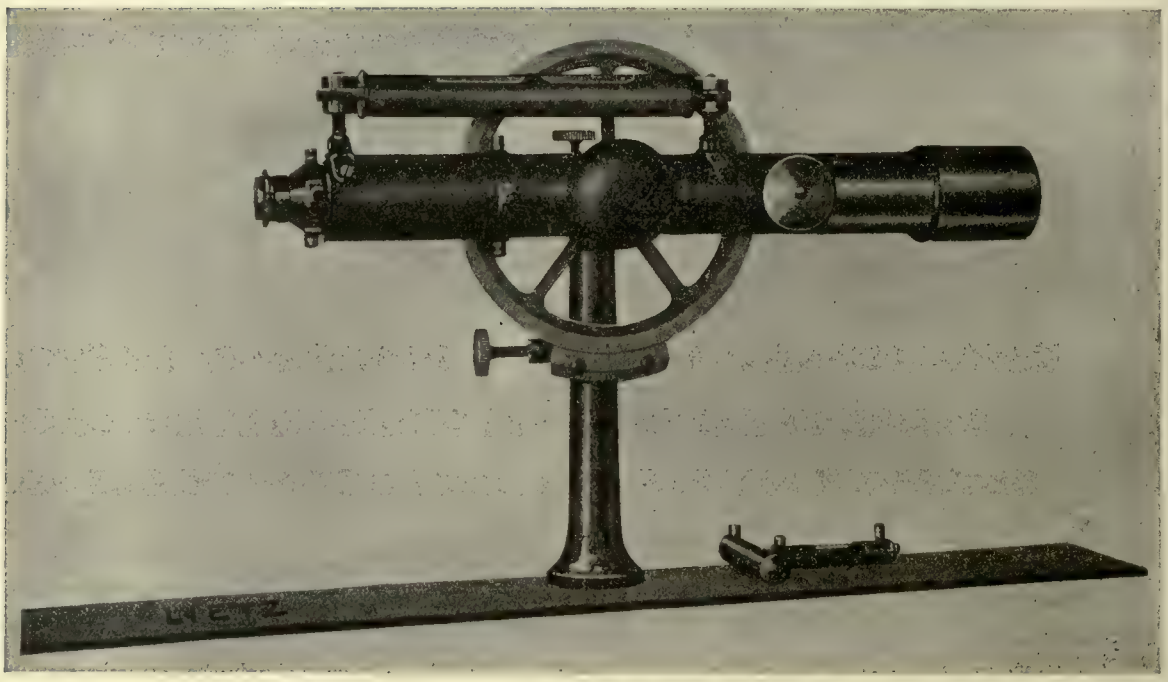


PLATE X

PLANE TABLE ALIDADE

Fitted with 11-inch telescope; magnifying power, 24 diameters. Revolving telescope (180 degrees) to check cross hairs; edge graduation on 60-degree arc graduated on solid silver. Blade beveled one side, 18 inches long, 3 inches wide, with two bubbles set at right angles; detachable striding level to telescope, clamp and tangent to vertical movement; fixed stadia hairs set 1:100; prism to eye-piece.

By either of these tripod connections the table (i.e. the board) may be brought into a horizontal position and also while remaining horizontal it may be turned in azimuth about its vertical axis. This corresponds exactly to the lower plate and its motion in the transit.

(243) The Alidade. (See Illustration, Plate X.)

The other part of the plane-table is the telescope set upon a standard that has for its base a broad substantial ruler made of metal and having its straight-edge parallel with the line of sight of the telescope; the telescope may be turned through an angle ranging between 40° and 60° in altitude and carries a vertical arc for measuring angles of elevation or depression. The ruler and the attached telescope and vertical arc constitute the alidade. It corresponds exactly with the upper-plate and motion of the transit (the alidade-plate). The alidade-plate of the transit has a graduated limb and verniers, which are wanting in the plane-table alidade, the angles in case of the latter instrument, instead of being measured, are directly graphed upon the plane-table sheet or map. For this purpose a sheet of suitable paper is fastened by means of clamps or thumb tacks to the drafting board.

The plate-bubbles of the transit have their counterpart in two tubes fastened to the ruler at right-angles to each other; or, in some instruments, a separate level upon a movable plate is supplied, by which the drafting board may be made horizontal and the vertical axis of the plane-table may be brought into verticality.

The telescope also has a bubble tube attached after the manner of the transit. In some instruments this bubble tube is detachable, as it is used only for determining the index error of the vertical arc, after which the arc itself is used.

Upon the ruler, or fastened to the drafting board, or apart from these, is a compass needle for determining the magnetic meridian and for orienting the board thereby. As a full circle or even a large arc is not essential for this purpose, a fragmentary arc of perhaps 5° or 10° either side of a mid-position marked zero (0) suffices; such a compass contrivance is called a declinator. The detached form of the declinator is preferable to a compass needle connected with the board or the ruler of the alidade as it can in its detached form be moved to any part of the map and readily brought into parallelism with any lines of the sketch. Therefore, it is made in a shape adaptable for this use; the needle, 4" to 6" in length, is suspended on a pivot in a narrow bronze box, 6" x 1" in size, with parallel sides and a glass cover to protect the needle. The needle may be lifted from the pivot when not in use to prevent wear on the pivot point.

It must be understood that the plane-table in its best form is not an instrument of precision; but, applied with care and with attention to the several means of approximate measurement, it will give results well within the limits of accuracy applicable in mapping. The attention of the student is particularly called to the following directions in setting up and using the plane-table, in order that the best results may be secured in the work.

(244) Observe that the tripod shall always be set firmly with its feet well driven into the ground and the legs should not be spread too far apart nor the board raised too high, as in either of these positions the table will be unstable for working or too high or too low for convenience in sighting and drawing lines.

Next in importance is the leveling of the board. This is done by means of the bubble tubes upon the alidade ruler or by means of the detached level as the case may be. The board is turned in azimuth in the leveling process in order to set the vertical axis truly vertical, the plane of the board being set perpendicular to this axis.

The edges of the ruler are made parallel to the vertical plane through the line of sight of the telescope, and in the better forms of alidade one edge of the ruler is placed directly in this vertical plane through the line of sight. Ordinarily it is sufficient that the lines of the drawing all be parallel to the line of direction of the lines of sight, but in some cases it is desirable to take sights for extreme accuracy, that are coincident with the vertical plane. The edge of the ruler that determines the line to be drawn should be set upon the point through which the line is drawn so that the pencil point in drawing the line passes through this point; as with any straight-edge it must be set off the point the thickness of the pencil point. In drawing lines, hold the pencil against the ruler's edge always at the same angle, otherwise the line drawn will be a curve instead of a straight line.

Place the alidade on the board in such position that the eye may be brought to the eye-piece of the telescope in sighting; or set the telescope objective end near enough to the farther edge of the board so that the point sighted may be seen when sighting an angle of depression.

Always be careful in sighting and shifting the alidade; and in drawing lines do not disturb the position of the board. It is easy to throw the board out of level and a slight twist may turn it upon the vertical axis.

There are several ways of making a survey with plane-table and we shall now proceed to give each of these in detail.

(245) **Traversing with Plane-table.**

This method, sometimes called progression, is similar to traversing with compass or transit. The plane-table is moved from point to point of the field and set up over each in some determined order; the lines are drawn by the ruler after it has been centered upon the point occupied and the telescope has been sighted upon the adjacent points of the field, - on the back point for a back-sight, on the forward point for a fore-sight. The distances are measured either by tape or stadia and laid off on the lines drawn, according to some determined scale; thus the angles and distances in each course are laid out directly in the field and the map may be embellished by any additional line or data required, - as, tie-lines to other points, details of houses, building, fences, streams, etc. These details are

usually located by either one or the other of the methods which follow: i.e., by radiation, intersection, or resection. (See Figure 101 to further illustrate this plane-table method.)

ILLUSTRATION OF A TRAVERSE IN DETAIL

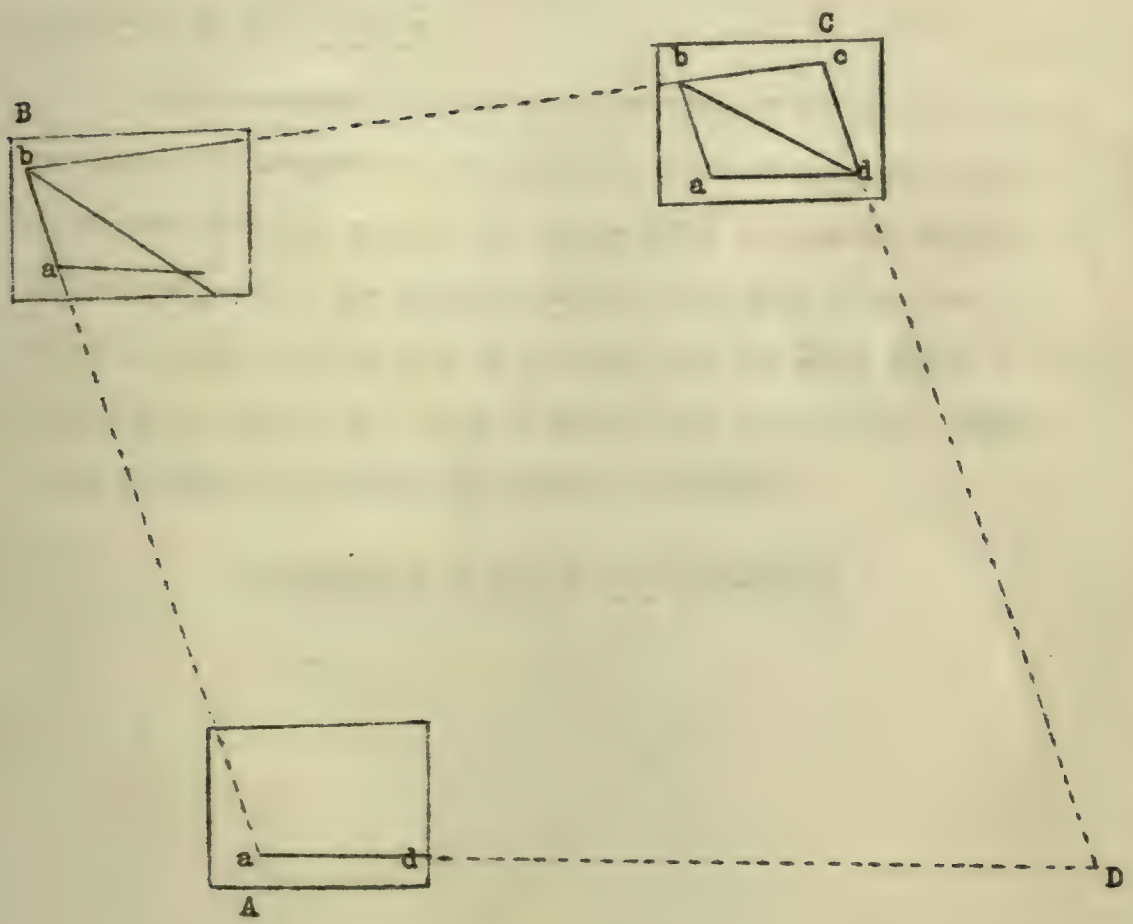
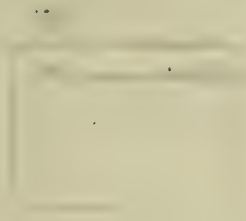
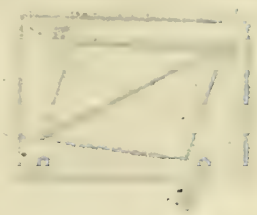


Fig. 101

Traversing. Draw ad representing AD to scale. Set up table at A centering a over A; with alidade on ad sight D





by turning the table in azimuth; with alidade centered on a sight B without moving the table but by adjusting the alidade; scale of ab equal to AB. Occupy B; back-sight on A, turning the table; fore-sight on C; scale bc; also sight D and scale bd. Move to C; back-sight on B; fore-sight on D, drawing cd. Occupy D; back-sight on C and check through ab

(246) Surveying by Intersection.

In this method one line of a traverse or field is carefully determined in length and its direction selected upon the sheet; it is then drawn to scale; the plane-table is then set up over one of the points, the alidade centered upon this point, the ruler in line with the line of the map, and the other point is now bisected by turning the table in azimuth to effect this. Figure which follows will further elucidate this method.

ILLUSTRATION OF METHOD BY INTERSECTION

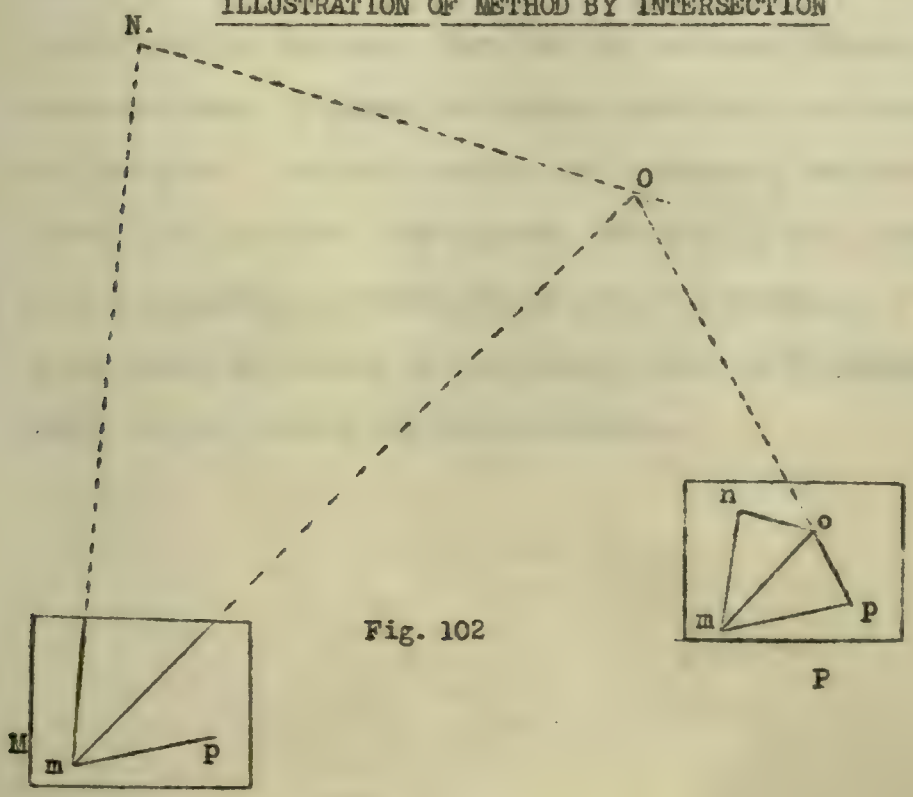


Fig. 102

Intersection. Draw mp to scale for MP in the field. Set up table at M, centering m over M. Place alidade on mp and sight P by turning the board in azimuth. Center alidade on m and draw indefinite lines toward N and O (or to any other points required). Move table to P; back-sight on M by turning the board. This orients the map. Intersect on N, O or other points, thus locating by intersection.

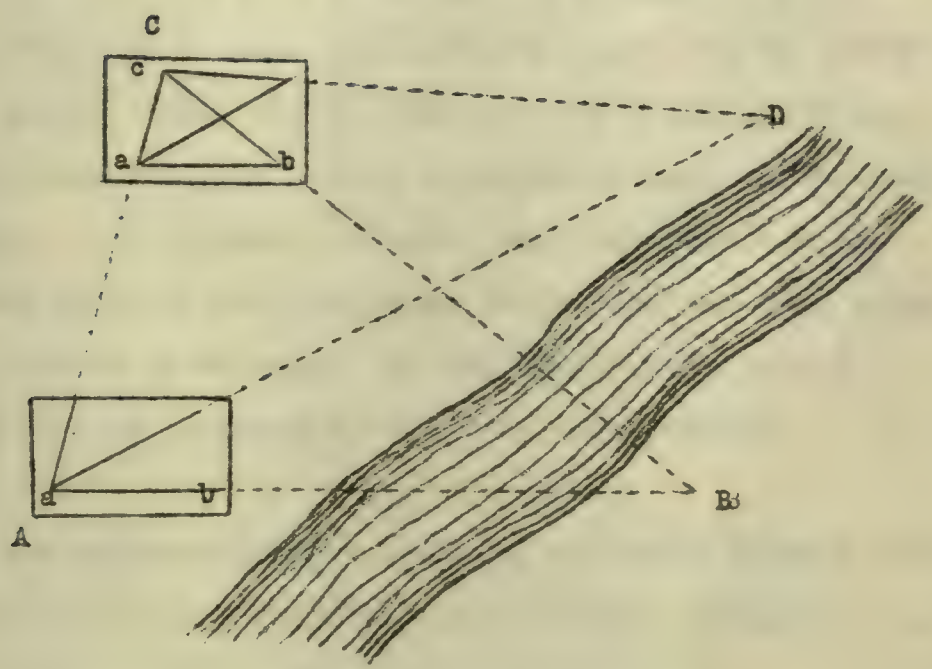
(247) Orientation.

This operation brings the table into such position that a given line or lines of the map are brought into parallelism with the same lines in the field; in the two methods above described the orientation was secured by backsighting in the manner shown. Orientation is sometimes also secured by drawing a meridian upon the map. Then the map is turned through the necessary angle to insure its correct position with respect to this meridian; for this purpose the compass or declinator is useful, and of course, the magnetic meridian is most convenient. In most plane-table problems the matter of orientation by any of the means applicable to the several cases is an important precedent to its correct and final solution.

(248) Resection:

A third method of surveying by plane-table is that of resection. This is especially applicable when one or more points of a traverse are inaccessible or when, for any reason, it is desired to avoid occupying such a point. In the Figure 103 suppose, for example, that a traverse ABCD is to be surveyed and the

Fig. 103



point B is situated as shown, on the opposite shore of a stream or body of water. Here if the side AB of the traverse is known in direction and distance, we may graph this to scale upon the map; then set up the table at A, centering a over A; orient by placing alidade on ab and bisect B, turning the board; then keeping the alidade centered on a draw indefinite lines to C and also to D (or to other points as desired); next determine the distance ac, scaling this on the map; move the table to C, setting the point c over the point C on the ground, as nearly as possible, orienting the table by back-sighting upon A through ca; now, with the alidade sighted on B, draw bc, or resect from B on C. The triangle bac on the map is proportional (i.e. similar and to scale) to the triangle on the ground BAC, for the line ab is to scale, the angle bar is constructed equal to its corresponding angle in the field, and angle acb is equal to angle ACB; hence one side and the three angles of one triangle are proportional and equal respectively to those of the other. We may now intersect upon D, drawing cd and similarly for any other points desired.

The resection method is specially applicable in the graphic solutions of the three-point problem and of the two-point problem which will be treated in the following assignment.

(249) Radiation.

A fourth method of surveying with plane-table is that of

radiation; it is synonymous with the method in compass or transit survey known as "the method by single set-up", which has already been explained in Assignment 15, on Land Surveying. By this method a point on the map is centered over a point in the field whose position is known and sights are taken to the various points whose distance and bearing (or azimuth) are required. The distance is measured by tape or, as is more common, by stadia. The line is drawn directly upon the map in the field; the angles are not themselves required as the mapping is complete without the numerical measure of angles, and the map is the chief consideration in most plane-table work. Since the mapping is done to scale the interior angles, bearings, or azimuths may readily be taken off by protractor, the lengths of the sides of the composing triangles can be used for computing areas where required or the planimeter may be used for the last named purpose.

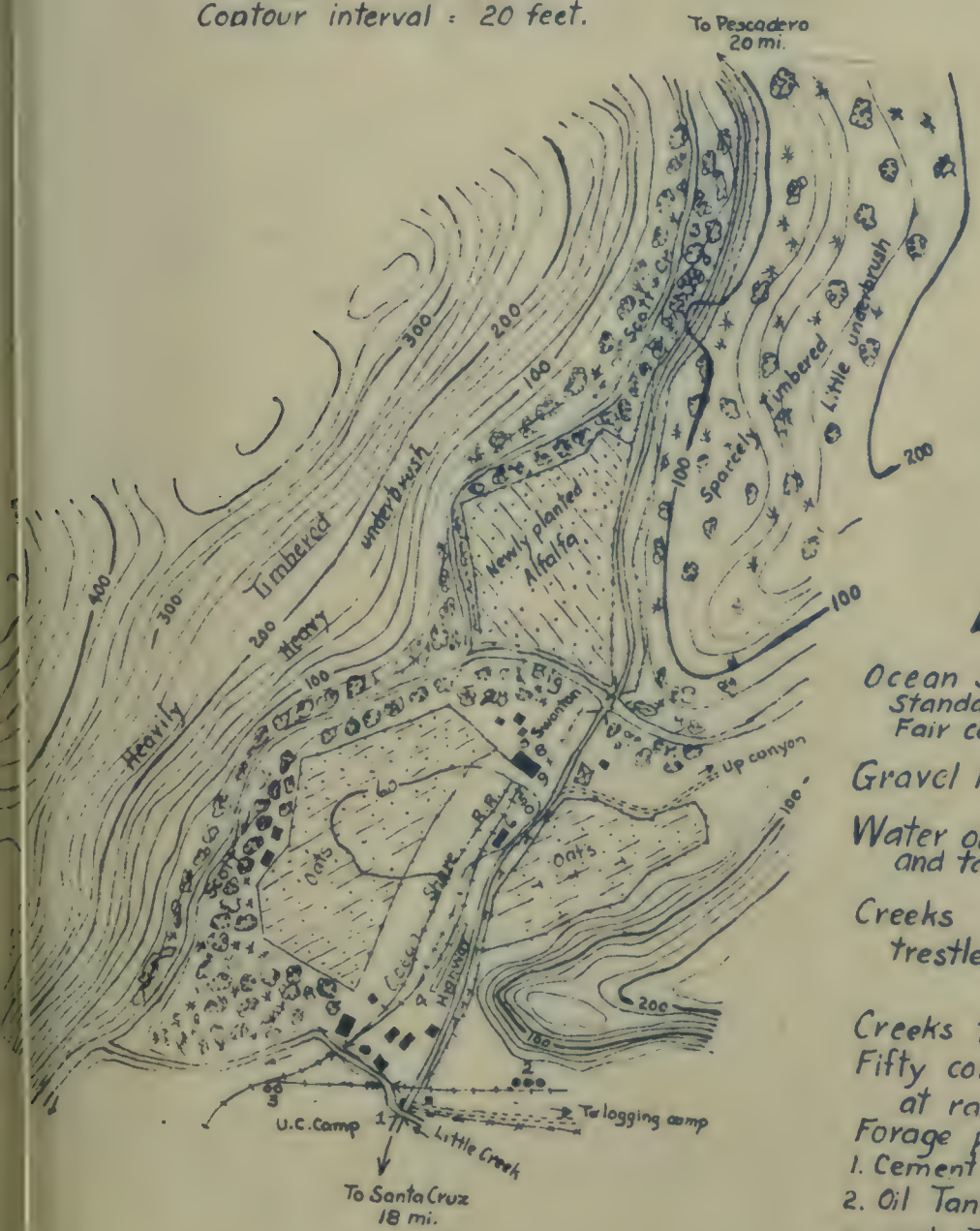
We append a small Military Sketch which will serve to illustrate the application of the plane-table to this kind of work. The sketch, very crude of course, shows how roads, streams, watershed, fence lines, bridges, etc. are sketched. The accuracy of such work depends upon the use to which the survey is to be put; in railroad or other construction work the measurements are carried to a higher degree of accuracy and many notes accompany the mapped survey.

All in all surveying by plane-table is useful because it is sufficiently accurate for the purpose required; it is cheaper

and, when records or refined work are not the object, it serves its purpose adequately; - many surveys are made for a certain definite object which, when accomplished, renders the more elaborate survey useless or obsolete.

Military Sketch
Swanton and Vicinity

Scale: 12 in. = 1 mile.
Contour interval = 20 feet.



LEGEND

Ocean Shore R.R.
Standard Guage
Fair condition

Gravel Road ~ Heavy traffic

Water obtainable in creek and tank at Swanton

Creeks crossed by wooden trestles and concrete culverts

Creeks fordable
Fifty cords wood obtainable at railroad terminal
Forage plentiful

1. Cement culvert
2. Oil Tanks
3. Water Tanks
6. Freight Depot
7. Steel Bridge
8. Small Vegetable Garden
9. Barn suitable for 10 horses

Tr - ABS

QUESTIONS OF ASSIGNMENT 34.

1. Compare part by part a plane-table with a transit. (Make replies brief).
2. Give the steps necessary in a plane-table survey of a four sided field, using traversing and radiation.
3. By means of data of your own choosing, show the relation between angle measurement of 1 minute of accuracy and lines to 1/100 ft. in accuracy and measurement secured in mapping to a scale of 1 inch equal twenty feet.

References:

Breed & Hosmer, Vol II. pp. 191-203.

Johnson, pp 117- 123

Raymond, pp 268 - 275

Tracy, pp 318 - 325

UNIVERSITY OF CALIFORNIA EXTENSION DIVISION

Correspondence Courses

Surveying 1B. Assignment 36. Mr. Swafford

PLANE-TABLE - THREE - POINT PROBLEM, Etc.

Foreword. This Assignment will deal with the further use of the plane-table, with a special application in the solution of the Three-point Problem and the Two-point Problem.

(250) The Three-point Problem:

A clear statement of the problem is as follows:

Given:- the location in the field of three points and their plotted positions on the map and the station in the field of the observer.

Required:- to plot this position of the observer upon the map. (The three given points may or may not be inaccessible).

Suppose that we have the three given points in the field,

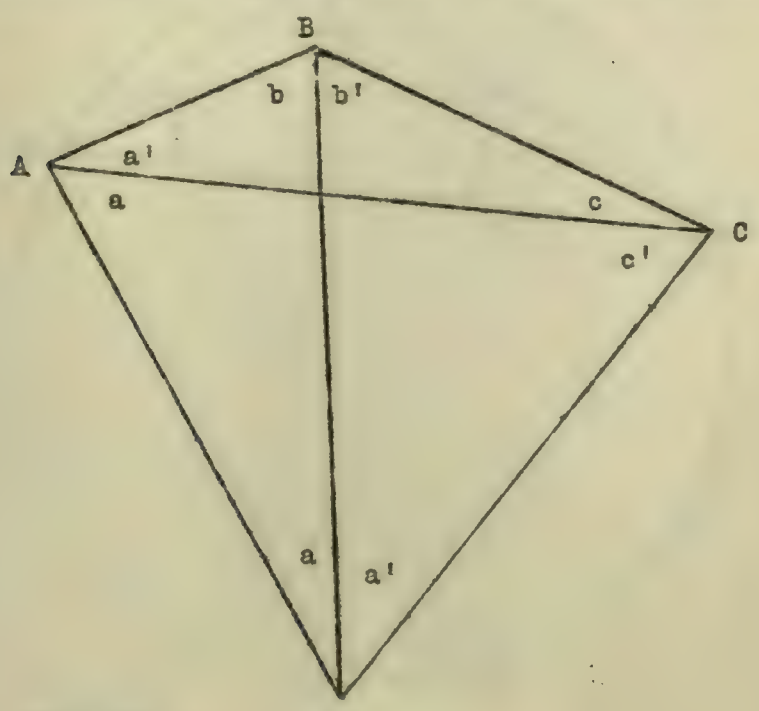
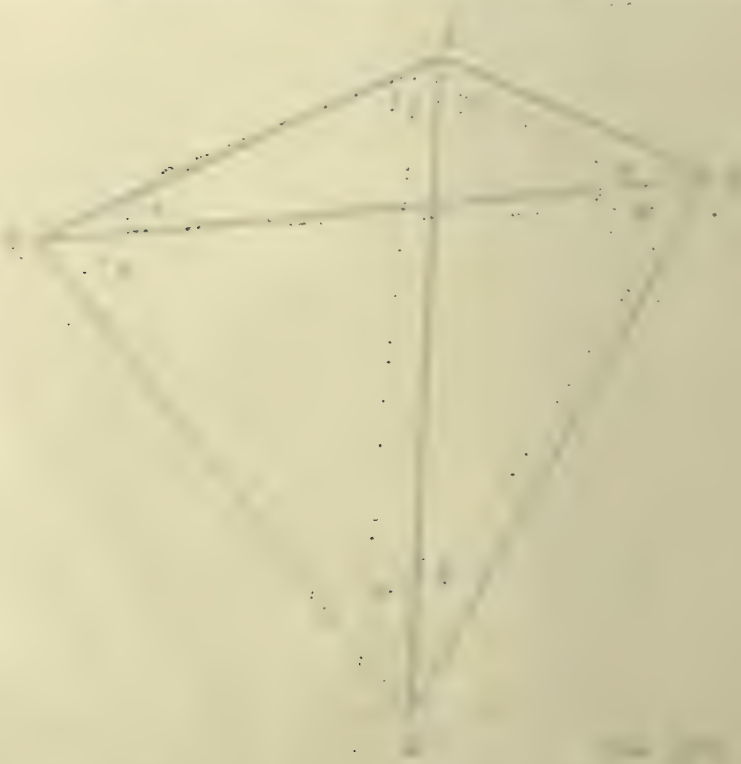


Fig. 104



A, a light-house; B, a church spire; C, a head-land; and D, which may be assumed as the location of an observer, as a boat at sea, or any location from which all points A, B, and C are visible but inaccessible. In the problem the distances A B, B C, A C are known; it is required to find the distances A D, B D, and C D. To do this it will be necessary to have the angles A D B and B D C; these may be had by taking the bearings of the several lines from D, or their azimuths, or by simply measuring the interior angles by one of several methods. Aboard a boat at sea the angles may be taken by sextant - usually two simultaneous observations being made, if this is possible. Or, on land a transit or even a compass may be used and the requisite angles or bearings be taken.

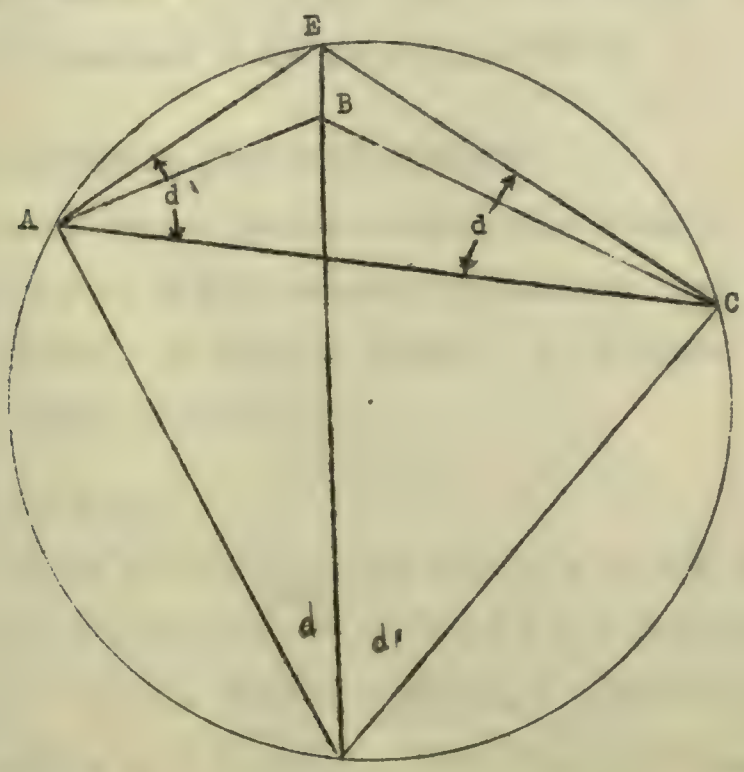


Fig. 105

If the angle ECA be constructed equal to angle d (= EDA) and angle EAC be constructed equal to d' (= EDC), the lines AE and CE will intersect at some point E which is on a diagonal of an inscribed quadrilateral AECD, AC being also a diagonal of the same quadrilateral; for ECA = ADE (both measured by the same arc AE), and ACE = d', (both measured by the arc EC). It is now possible to locate D since it is upon the circle through AEC and also on the prolongation of EB, i.e. at D, which is the intersection of the diagonal with the circumference. Hence the problem is graphically solved.

The problem may also be solved trigonometrically, but the plane-table method is essentially a graphic method and the construction given above is necessary only for the purpose of explaining the principles of the plane-table method.

(251) Methods of Solution of Three-point Problem:

There are several methods of plane-table solution of the three-point problem of which we shall explain four: 1. The Protractor method, 2. Bessel's method, 3. Triangle of Error method, 4. Llano's method.

1. Protractor Method:

Set up the plane-table at the observer's station, and center a three-arm protractor on the point d (the point upon the table directly over the observer's station, D). Direct the arms of the protractor to A, B, and C respectively, setting off the

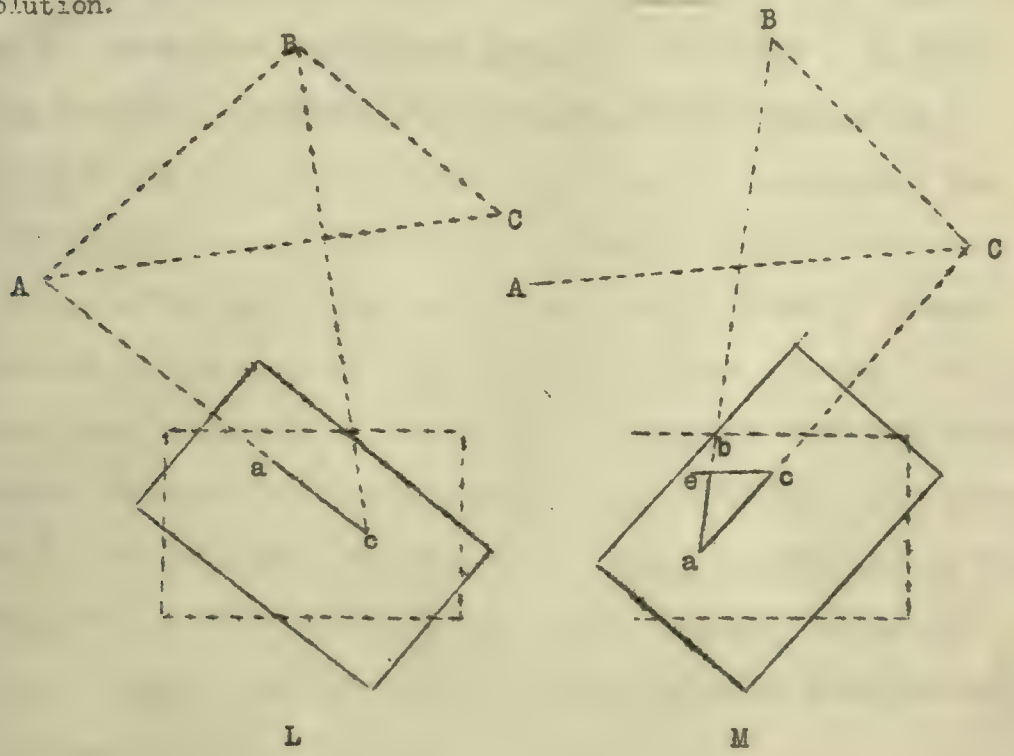
angles A D B. B D C. The points a, b, c, are then plotted upon the map in the relative positions of A, B, C in the field with distances according to the scale of the map. Orient the board approximately by estimation. Place the protractor so that the three arms shall pass through the points a, b, c in their respective positions, sighting through a on A; check by sighting through b on B and also through c on C. To do this may require a slight adjusting of the board, for the approximate orientation only partially accomplished the desired position; when the three arms pass through the three points a, b, and c and sight A. B, C, the protractor center must be at d, the plotted position of D.

This is known as the mechanical method of solving three-point problem. Instead of a three-arm protractor, a piece of tracing paper or tracing linen may be used. Draw lines upon the tracing paper by means of the alidade by first assuming a point on the tracing paper and then intersecting upon A, B, C; then secure the proper orientation by placing the tracing so that the points aA, bB, cC are in line. A point may now be pricked upon the map through the assumed point on the tracing. For rapid approximate work, this is a useful method, but should not be relied upon for great accuracy.

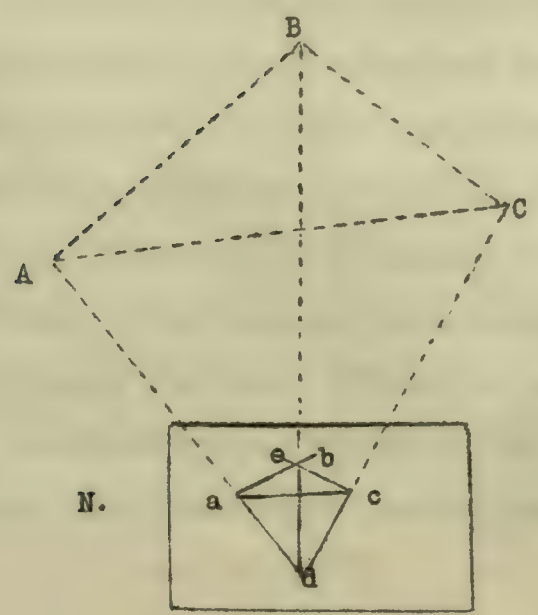
2. Bessel's Method.

This method consists of constructing the angle formed by the lines from D (the position of the observer) to A, B, and C, drawing these directly in the required position upon the map and

thus determining the fourth vertex of the quadrilateral. A study of the following figures will make this clear; the three figures L, M, N, represent the plane-table in the three steps of the solution.

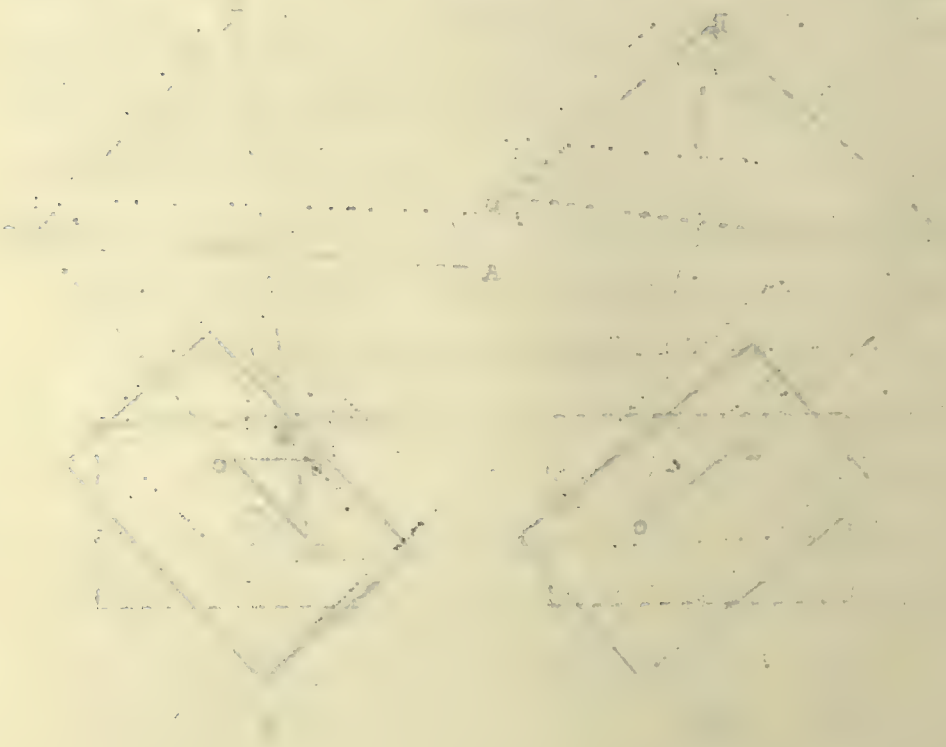


L M



N.

Fig. 106



Set up the plane-table at D; place the alidade upon ca and sight A through a by turning the table; clamp the board and with alidade centered on C, sight B and draw the line from c toward B of indefinite length; this step is shown in Figure 106 at L. Again place the alidade upon ac and sight C by turning the table; clamp the board and with alidade centered on a, sight B and draw the line from a toward B to intersect the previous line so drawn at e in the figure; this step is shown in Figure 106 at M. Now draw a line through b and e, place alidade on this line and sight B; this orients the map. Resect from A through a, or from C through c; both resections should intersect in a common point on be prolonged; this point is d, the point upon the map representing D, the position of the observer. A circle through aec will also pass through d; i.e. aecd are the vertices of the inscribed quadrilateral discussed above.

(252) The Triangle of Error Method.

This is also known as Lehmann's method, as an exhaustive investigation and discussion of this method has been prepared by Prof. H. Lehmann of the U.S. Geological Survey, in which it is extensively used with gratifying results. By its use the observer's position may be quickly and accurately mapped, and its use is readily acquired by the trained plane-table topographer.

The solution by this method (Lehmann's) is a simple application of resection from the signals in the field through the plotted positions of the same points upon the map. As in the

other methods, the solution is practically secured when the map has been correctly oriented. You should always bear in mind, therefore, the necessity for correct orientation and the means of securing this.

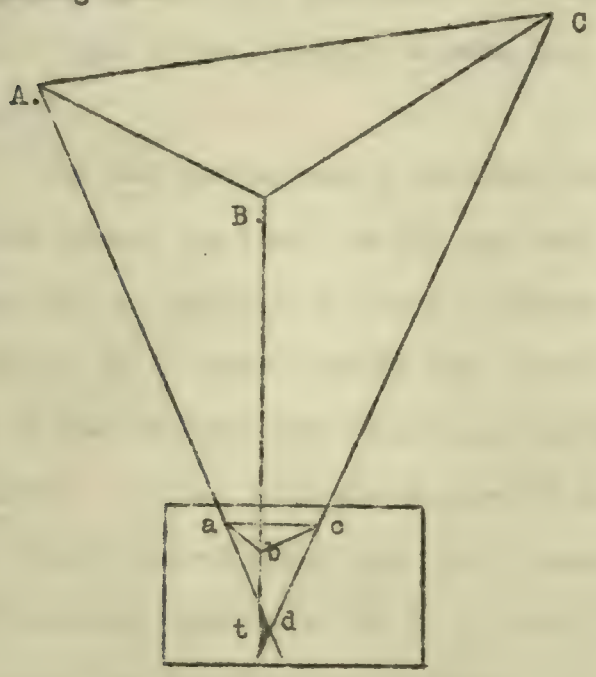
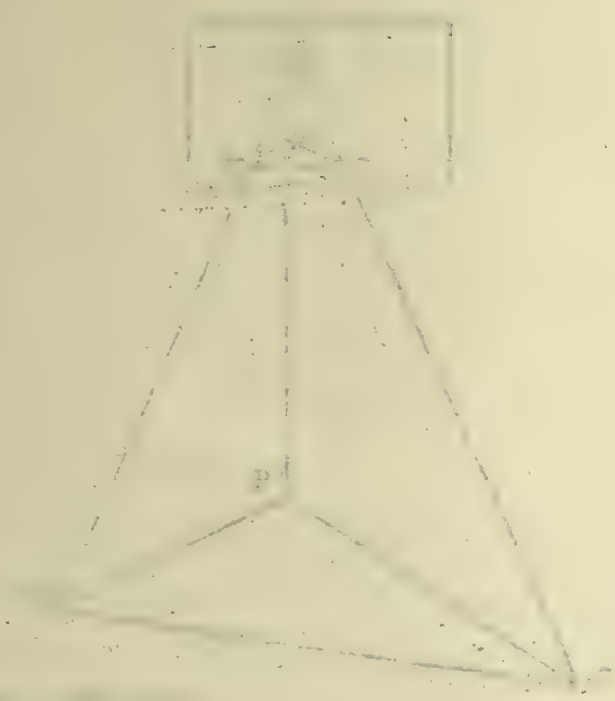


Fig. 107

Figure 107 shows the plane-table set for determining the position of observer and plotting it upon the map. The resection has resulted in producing the small shaded triangle at t; but if the map had been correctly oriented, i.e. had ac been parallel to AC (from which it would follow that ab was parallel to AB and that bc was parallel to BC), then by the laws of geometry the lines through the vertices of the two triangles ABC and abc must intersect in a point; the existence of a triangle instead of a point is evidence that correct orientation has not been secured.

Hence, to secure correct orientation; turn the board in azimuth in that direction which will tend to dimish the triangle t, (called triangle of error) until the resection lines have a



common point of intersection. The position of the table in this illustration is such that the board should be turned counter clock-wise in order to accomplish this result; in other words the point d (plotted position of the observer) will be found to the right of the triangle in some such point as d in the figure.

You can readily verify the fact that if the point d were to fall beyond the limits of the map that this method or any method fails. Again if a circle is drawn through the signals in the field A, B, and C, called the "great triangle", any station of the observer upon this circle in the field becomes indeterminate - i.e. with the observer's station at any point on this circle the resection lines will intersect in a point as the quadrilateral formed with A, B, C, and D as vertices would constitute an inscribed quadrilateral.

It may be seen that in Figure 107 , the point D is necessarily outside of the "great circle", as the circle passing through points A, B, and C is called.

The solution is said to be strong when the observer is in certain positions; these are well outside of (or within) the great circle, or when the observer is within the great triangle which is the triangle formed by the three signals. Also the solution is strong when the observer's station is nearer to B (the middle signal) than to A or C. Figure 107, therefore, showing a specially strong solution.

Since two or three attempts to determine d may be made by trial, this method is known as "the triangle of error method". The student will find an admirable treatise on the plane-table by D. B. Wainwright, published by the U.S. Government, Bureau of Publications, Washington, D.C.

Llano's Method.

This method is essentially geometric and appeals to many plane-table topographers as a direct and simple solution, consuming little time and giving results adequately exact for most purposes.

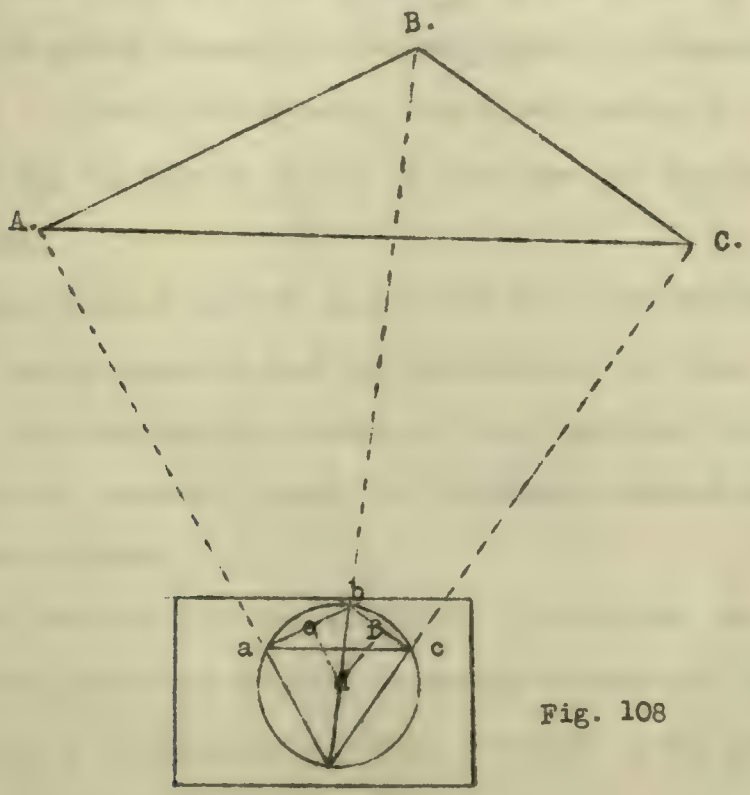


Fig. 108

The plotted positions of a, b, c are drawn upon the map. Bisect ab and draw a perpendicular; place alidade on this perpendicular

bisector; turn the board and sight B; resect from A through a and where this line cuts the perpendicular mark e; draw the perpendicular bisector of bc; place alidade on this line and turn the board sighting B; resect from C through c and mark f; with e and f as centers and ea and fc as radii draw arcs intersecting each other; the intersection of these arcs is d, the plotted position of the observer. Verify this by centering alidade on d and sighting A, B, C. NOTE:- The position of d may also be found by drawing a line from d perpendicular to ef; this point is as far from ef as the point b is.

This method depends for accuracy upon the distance between e and f; when this is small, the determination is inaccurate; if the two points e and f coincide the problem is indeterminate.

The choice of methods is governed by the conditions in each case and by the means at hand for constructing the lines in the figures. Practice and the consequent skill acquired by the topographer will generally enable him to choose a method suited to the case in hand.

All methods fail when the point d falls off the map, the only remedy is for the observer to change his position to a nearer station, or in any case one more favorable to the proper and accurate solution.

(253) The Two-point Problem.

The observer's position may be determined when only two

points in the field instead of three are given. This is known as the two-point problem, and its solution rests upon the assumption of a third point located by the observer. This usually is accomplished by setting up a line (of the character of a base-line) by locating two points for observation, one of which is the point sought. It is evident that an understanding of this problem is quite essential to the efficient topographer, since cases arise when it is necessary to use it. A statement of the problem is as follows:

Given - the location in the field of two points and their plotted positions on the map and the station in the field of the observer. Required - to plot the position of the observer upon the map.

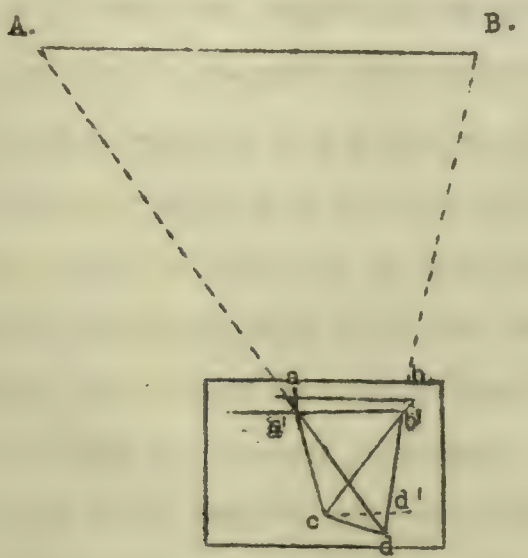


Fig. 109

Referring to Figure 109, in which A and B are the points in the field and T is the plane-table on which the points a and b have been plotted to scale and the map supposedly oriented. Assume d, and intersect on A and B

through a and b respectively. Then move table to C and intersect on A and B, through a and b. The intersections at a' and b' with c and d will form a quadrilateral of the proper form but incorrectly placed. Hence turn the board through the angle dcd' and resect again for d (or c, if desired).

If a point can be occupied in line with the two given points A and B, the table may be set up at this point, which we may call P (on map p), and the board oriented by placing the alidade on pba and sighting BA in line; then direct the alidade toward some point Q in the field and draw a line pq; finally occupy Q, backsight on P with alidade on qp, thus orienting the board. The point q is now located by resecting from A and B through a and b. This is a quick and satisfactory method practiced by topographers under the conditions indicated above.

(254) You should understand that in most plane-table problems the whole board is regarded as a point, and, when compared with the large areas over which such problems extend, it is accurate enough to regard it thus. It is well to realize too the similarity between the plane-table parts and those of the transit. The alidade corresponds with the alidade plate of the transit; the board, with its movement in azimuth, corresponds with the lower plate of the transit, etc. Follow out this comparison and note the completeness of this agreement or similarity. Realize, too, that while the transit is an instrument of refinement and precision, the plane-table is often the more economical and sufficiently trustworthy as to results obtained by its use.

QUESTIONS ON THE PLANE-TABLE

1. State the several adjustments of the plane-table and compare them with their corresponding adjustments in the transit.
2. Is it necessary that the line of sight of the alidade telescope and alidade ruler's edge lie in the same vertical plane? Give reasons for your replies.
3. Compare the four methods of plane-table surveying with equivalent transit methods for the same purposes.

REFERENCES:

Breed and Hosmer Vol. II, pp. 191 - 222.

Tracy 318 to 336

Johnson - Solutions of Three-Point Problem p. 278 et seq.

Raymond - Solutions of Three-Point Problem p. 291 et seq.

Bibliography

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Diamond - Applications of Three-Point Problems p. 283 et seq.

Johnson - Applications of Three-Point Problems p. 276 et seq.

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Correspondence Courses

Surveying-1B Elements of Surveying Mr. Swafford

Assignment 36

MINE SURVEYING

Foreword: A brief treatment of the subject of Mine Surveying, including the instruments and the methods employed and some of the problems peculiar thereto, will constitute the scope of this assignment.

The terminology and nomenclature used by miners and others connected with mining need not be set forth at length in this connection; but a few terms and names are here defined in order to avoid unnecessaty details and circumlocution in the introductory part of the subject.

The essential difference between the surveying subject already dealt with in this course and mine surveying lies in the fact that in addition to the usual problems we have those relating to underground areas, shafts, tunnels, and so forth. Many of the methods and the instruments used in the work peculiar to the usual surface surveys are found unsuited or wholly inadequate to the proper or convenient accomplishment of underground work; while many of the problems of mine surveying differ materially in their nature from those of general surveying as already treated.

(255) A Few Terms Defined: A shaft is an opening from the surface downward into a mine; it may be vertical or more or less inclined, and is not always straight. This is used for entrance of workmen

and others into the mine and for introducing machinery and for removal of ore or other material. For these purposes a bucket or cage operated by cables and hoisting engine is used.

A tunnel is an opening into a mine and usually extends horizontally with two openings; i.e., it passes through the hill or mountain into which it penetrates, although the term is loosely used to indicate a more or less commodious ingress into the mine for purposes similar to those of a shaft.

An adit is still another form of opening from the surface, usually at the general level of the diggings and hence is often called an adit-level. Unlike the tunnel the adit only opens into the mine by one entrance and has no other outlet.

On hillside diggings the adit or tunnel is the more logical construction of entrance, it is generally more easily and cheaply built and permits of operations that require a minimum of lifting.

The terms floor, roof, and wall are about synonymous with these terms when applied to the corresponding parts of houses and may be understood without special definitions.

Collar is the term used to indicate the timbers around the opening of the shaft. At the entrance of a tunnel or adit the name frame and timbering is sufficiently descriptive.

Levels are the series of horizontal workings at successive elevations or depths.

A connection is a short passage connecting two or more parts of the workings on the same or different levels.

A winze is a connection of the nature of an underground shaft that, while it may connect two or more levels, has no exterior opening.

A vein is a stratum of ore; usually this is imbedded in rock of a different sort from the ore itself, and the workings are usually directed along the course of the vein.

Dip and strike are terms used to define the course of the vein; dip means the angle of inclination of the rock plane to the horizon; strike is the bearing (or "cross-country" direction) of this rock plane in a direction at right angles to the dip. A horizontal passage following the vein is called a drift.

(256) Mine surveying is from its nature divided into two branches differing more or less widely, in that one has to deal with the underground work, while the other deals with surface measures. Traverses are run on the surface and underground to locate and define the property lines above and below ground, to give bearing and distances of tunnels, shafts, connections, drifts, etc. Depths, slopes, and azimuths are determined with the greatest accuracy possible, and these must be connected with the surface survey by lines measured in angle and distance with careful precision. To accomplish this a meridian (true or magnetic) or a line of known azimuth is carried from the surface into the underground works. The methods for doing this will be given further on in this assignment.

A vein is a body of rock usually unlike the rock of the country, while it may connect two or more layers, has an exterior opening.

A vein is a structure of rock usually unlike the rock of different sort from the ore itself, and the workings are usually directed along the course of the vein.

Dip and strike are terms used to define the course of the vein; dip is the angle of inclination of the vein plane to the horizon; strike is the bearing (or "compass-course") direction this rock plane in a direction at right angles to the dip. A horizontal passage following the vein is called a drift.

Mine surveying is work the nature of which is somewhat different more or less widely, in that one has to deal with the underground work, while the other deals with surface measurements. Surveys are run on the surface and underground to locate and define the boundary lines of the mine and below ground, to guide the miner and direction of tunnels, shafts, underground workings etc. that are, and ultimately are determined with the greatest accuracy possible, and these must be connected with the surface survey by lines measured in angle and distance with careful precision. To accomplish this a permanent (this or another) base line is carried from the surface into the underground workings.

In the low underground passages in mines we meet conditions that differ greatly from those found on the surface. First of all the light is much fainter, often amounting to total darkness, except for the artificial illumination which is mostly poor and always insufficient for surveying purposes. Therefore, lights are used for signals and suitable lanterns for reading rods, graduated circles, and for recording notes and reading maps, records, etc.

Since the passages have low roofs, often only a few feet in height, it is necessary that rods, tripods, and other instruments should be short, or, as in case of tripods, of the extensible pattern. In some cases it is convenient to mount the transit head upon a bracket screwed into the wall or to place it upon a trivet, a sort of very low support having three feet, hence the name.

The presence of tracks upon which cars are run for transporting the ore, or the travel of men and animals over the floors of the passages, renders the setting of points thereon impracticable in most cases; hence hubs, points, and even monuments are placed if possible in the roofs or upon the walls of rooms and passages. This necessitates the suspending of plumb-lines, not from the transit, for example, but above the transit which is set up underneath, the point of the plumb bob centered over a point countersunk in the telescope axle.

Plummet lamps or candles may be used for signals; or a screen of paper or cardboard makes a good mark when illuminated

by a suitable light. The transit should be fitted with a reflecting shade, so that light from a torch or lamp may be reflected down the tube for illuminating the cross-hairs in the telescope.

(257) The transit for underground work should be of a substential but light pattern. It should have a large objective and a low magnifying power, as sights are usually short and much light rather than magnification is preferable. Either the erecting or the inverting type may be used, the choice depending upon the user, but the inverting type possesses many advantages.

A full vertical circle is essential as the instrument is frequently plunged. The vertical circle should be well protected by a cover guard, hence double verniers are convenient. As it is often required to sight vertically (or at great inclination) either upward or downward, the transit should have either a top telescope or a side telescope to enable the transitman to look past the plates, which is impossible with the ordinary transit in case of greatly inclined sights. The instrument should also be fitted with a prismatic eye piece for use with large vertical angles.

(258) The top telescope above mentioned consists of an auxiliary telescope attached over the top of the main telescope at a short distance from the main telescope equal to the radius of the horizontal base plate and a little more to permit of sighting past the plate. The auxiliary is attached best by two short columns connecting the two telescopes, but several forms of attachment are

made.

The side telescope is also an auxiliary telescope fastened to the side of the main telescope but usually in the case of this latter auxiliary it is screwed to the axle of the main telescope and rigidly fixed thereto, so that it may always turn with the main telescope, both remaining in the same plane with the horizontal axis. It is distant enough to clear the base plate or any projecting screw, and is properly counter-balanced by a weight attached to the axle at the opposite trunion.

(259) Measuring angles when sighting with the top telescope may be understood by the following consideration:

The top telescope and the main telescope are fixed in the same plane with the vertical axis of the instrument, the lines of sight of the two telescopes are parallel, and they are at a definite known distance apart, called the eccentricity; hence, all horizontal angles measured by use of either the auxiliary or main telescopes are the true horizontal angles; vertical angles are different in the two cases, the angle of error in reading with the top telescope being equal to the angle whose sine is the distance of a point from the instrument (horizontal axis) divided into the eccentricity of the telescope.

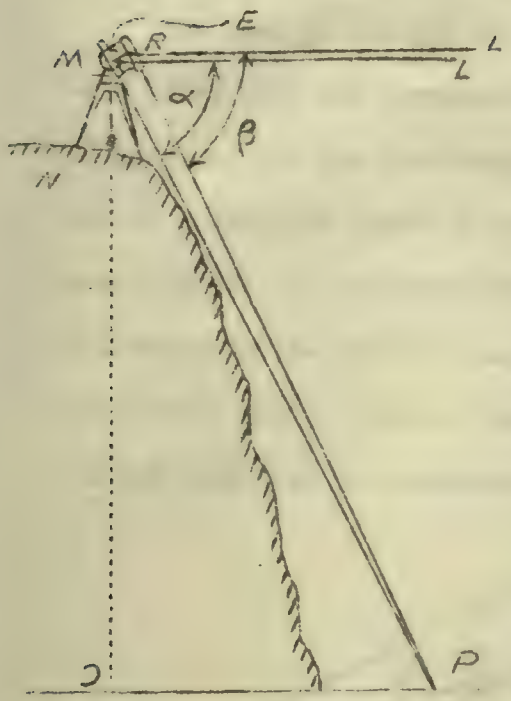
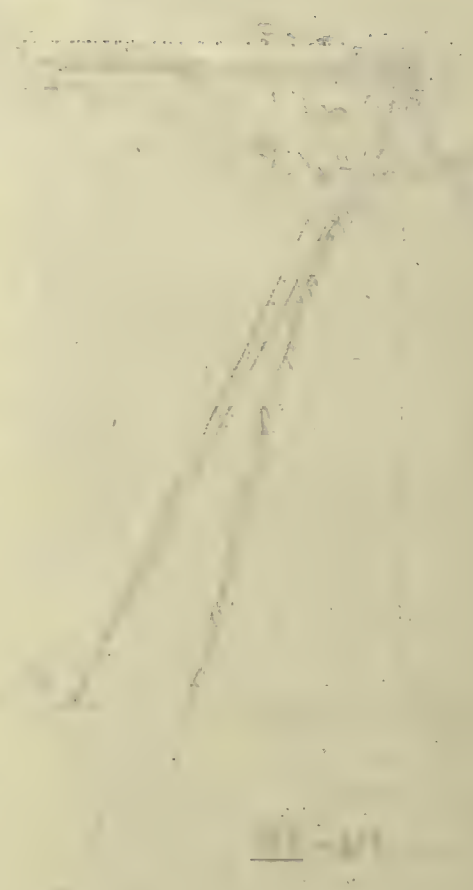


Fig. 110

Fig. 110 shows a transit set upon an eminence from which it is desired to measure the angle of depression by means of the top telescope, and to compute the height N (N) and the horizontal distance OP.

Sighting the point P with the top telescope, read the angle α on the vertical circle; measure the slope distance MP. But this gives a false angle of depression inasmuch as the top telescope must be depressed more than the main telescope in order to sight P, and the angle LRP is the true angle of depression which is greater than LMP; hence LRP - LMP is a correction angle to be subtracted from LRP. This angle has for sine, E (the eccentricity) divided by MP (the distance from the horizontal axis to point P). Now angle MPO is equal to the true angle of depression and OP = MP.cos MPO and OM = MP.sin MPO.

To facilitate computations a table of values for sin $(\beta-\alpha)$ at varying distances MP may be compiled. The eccentricity E being a constant, the sine of the correction angle varies inversely as the slope distance MP.



(260) The following considerations will make clear the use of the side telescope in measuring angles.

The side telescope and the main telescope are fixed in the same plane with the horizontal axis of the instrument, the lines of sight of the two telescopes are parallel in this plane, and are at a definite known distance apart (the eccentricity in this case); hence, all vertical angles measured by either the main telescope or the side telescope are true vertical angles; but horizontal angles read by the side telescope are false angles of azimuth and must be corrected as follows:

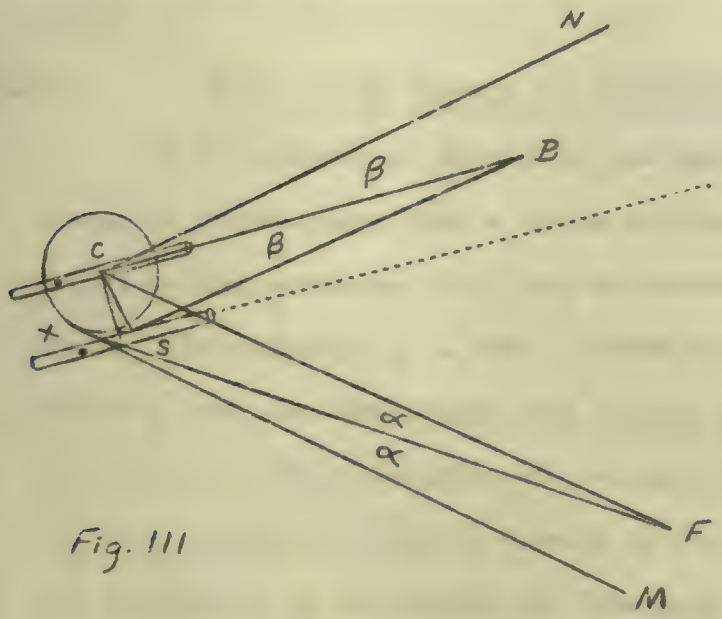
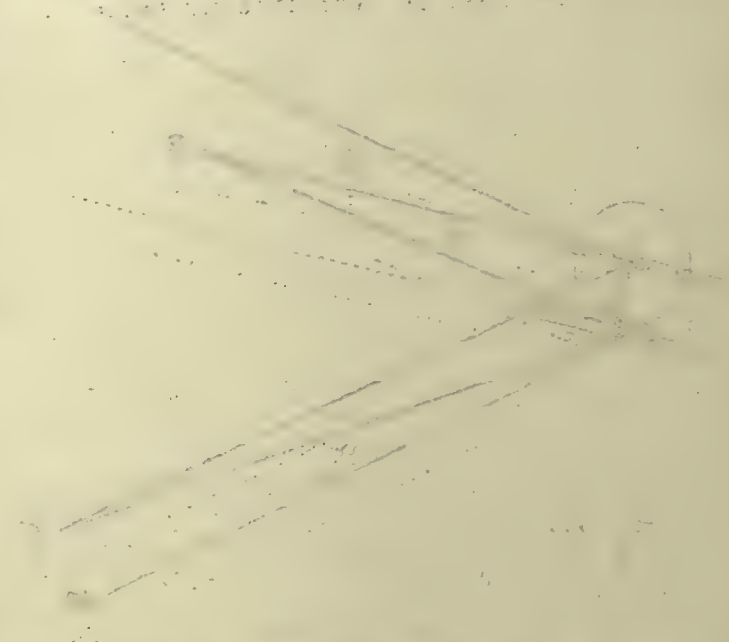


Fig. 111

Fig. 111 shows how azimuth may be corrected when horizontal readings are made with pointings of the side telescope. Sightings are made on B and F with the auxiliary telescope, but the angle required is BCF; as the angle turned in reading with the auxiliary is BXM it is plain that this value is increased by SBC ($=\beta$) and diminished by FXM ($=\alpha$); hence the true angle at vertex $\overset{C}{\wedge}$ is FXM - β + α. Here β = angle whose tangent is



E + CB, and α = angle whose tangent is E + CF, E being the eccentricity. When CB = CF no correction is required.

(261) Solar observations for meridian are best made by direct observation upon the sun, although the usual forms of solar attachment may be used. But the combined solar and top telescope attachment is not recommended.

The compass in mine surveying is of little use as an instrument for taking azimuths or bearings. The presence of iron ores, machinery, rails, and lighting and power lines for electric energy will practically preclude the use of the compass in this sort of work.

(262) RELATION OF SURFACE AND UNDERGROUND SURVEYS

For the purpose of connecting the surface and underground areas, it is necessary that a common meridian be carried from surface into the workings. This may be accomplished by one of several means. The meridian, or a line of known azimuth, may be carried directly into the underground area through a tunnel or an adit; to do this, it is only necessary to prolong a line of the surface into the mine through the adit or tunnel, or to carry a line of convenient bearing out of the tunnel and determine its bearing or azimuth with respect to some known line of the surface survey.

The method of carryihg a meridian into a mine through a shaft will depend upon the nature and size of the opening and possibly too upon the depth of the shaft. Plumb lines may be dropped

from two points upon the collar and the bearing of the line connecting them determined; this is practicable when the shaft is verticle, so that the plumb lines hang free of the walls, and when the opening is wide enough to allow of a line of sufficient length between the two points.

In case there are two shafts the latter difficulty is removed; points may be established at the bottom of each shaft by plumbing; the azimuth of the connecting line on the surface is then the azimuth of the line joining the two points below.

(263) A transit with auxiliary telescope may be used for the purpose of carrying a meridian into the mine through a shaft, and in case the shaft is inclined, this is the most feasible means.

Set up the transit at a convenient place at the mouth of the shaft; backsight on a suitable point on the surface with telescope plunged; right the telescope and set a point at the botton of the shaft, if possible, as far as practicable within the mine. Read the angle (deflection, as described below) and calculate the azimuth. Occupy the point below and set off the back azimuth sighting the point at the collar, and from this position lay off a line of convenient length in the mine, determining its azimuth.

Too great care cannot be given to the matter, as much depends upon the correctness of the line - all lines of the underground workings are connected therewith and a small error may be multiplied or at least repeated again and again.

(264) The distance down a shaft is measured by tape; if inclined, by supporting the tape at points along the inclined side; or if the shaft is vertical, by suspending the tape and putting the same under tension by hanging a known weight upon it, If the tape hangs free the tension of the tape will be $W+wl/2$, where W is the attached weight, w is the weight of a linear foot of tape, l is the length of the tape under tension. The tape may now be compared with a standard by supporting both tapes throughout their length and applying the tension above determined. Very deep shafts may be measured by means of a piano wire, which is afterwards compared with standard in like manner.

(265) Many measurements of distance are necessary or conveniently made from the head of the transit instead of from a point below the plumb bob and care must be taken to secure the correct measure. Again, since many points are chosen upon the roof or walls of rooms and passages, it is frequently necessary in taking angles of slope to such points to reckon the "height of point" as well as the height of instrument. The H.I. may be positive or negative depending upon its position with respect to the transit; if the point from which the H.I. is taken is below the transit, it is positive (+); if above the transit, i.e. measured downward, it is negative (-). Likewise, the "height of point" (H.Pt.) is positive when measured from above downward and negative when measured upward. Due regard to these quantities and their signs is of special import in reading angles of slopes.

(266) In running traverses at surface and underground they should be connected by starting at a common initial point and, if possible, closing upon a point common to the two traverses. The two traverses thus run will be open traverses but the closing line of one will be identical with the closing line of the other; hence, by computing this closing line for each of the two traverses a reliable check may be secured. In case the two traverses which are run from a common initial point cannot be closed on the same point, then the closing line of each traverse is comupted and from the data thus secured a closing line between the two is then also computed, but it is evident that no check is possible in this case.

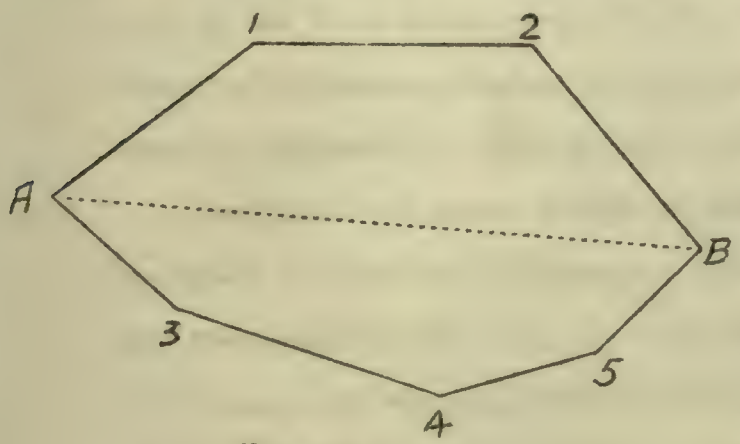
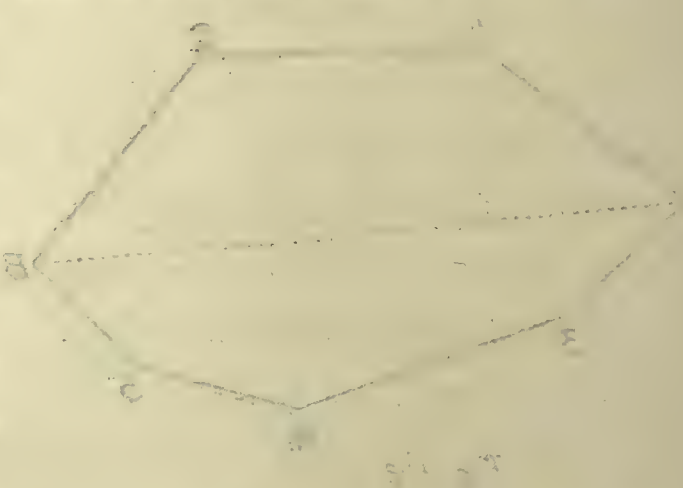


Fig. 112

Fig. 112 illustrates a surface traverse, A12B and an underground traverse A345B, beginning at A and closing on B. Here it is required to find the distance AB. This can be done by finding the missing part, AB and its bearing (See Assignment 18, on supplying missing data). The traverse A12B furnishes the necessary data, as also the traverse A345B; the value of AB by the two routes should check.

To illustrate any case where the common point of closing is wanting, see Fig. 113. Here the line MN is computed for the surface



traverse M12N and this line is then taken as a part of the underground (incomplete) traverse M3450NM - and the line NO computed by the method of supplying omissions. In this case, however, there is no check upon the work and all lines and bearings must therefore be measured with scrupulous exactness.

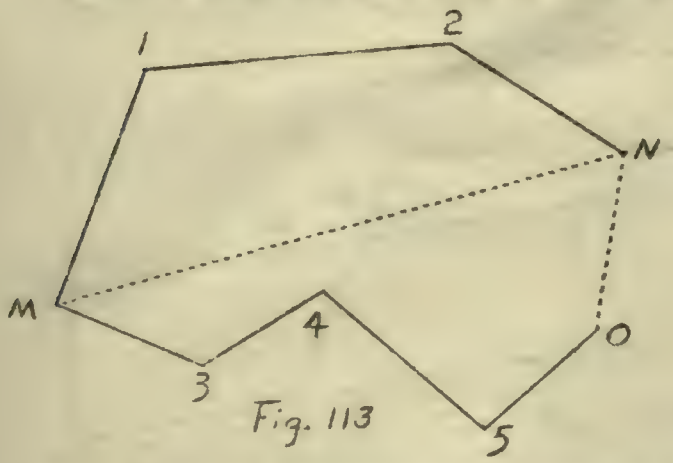


Fig. 113

(267) Mining Claims: The surface extent of mining claims is defined by the U. S. Government, and these limits must not be exceeded by claimants, although the limits, less than these, are sometimes regulated by local (State) laws.

Coal mine and placer claims are 20 acres in extent, by U. S. Government regulation, and mineral (lode) claims are defined as 1500 feet along the strike of the lode, either in a straight or broken line, and extending 300 feet at right angles to this line on both sides. The end boundaries are required to be parallel to each other.

The following diagram, Fig. 114, will further show the form of a mineral claim as defined by U. S. Government laws, for regulating same:

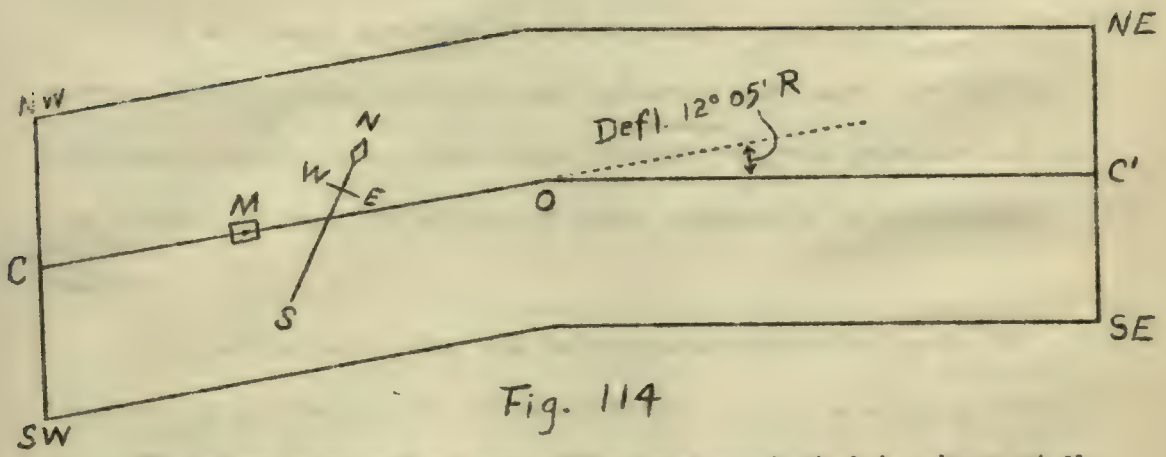
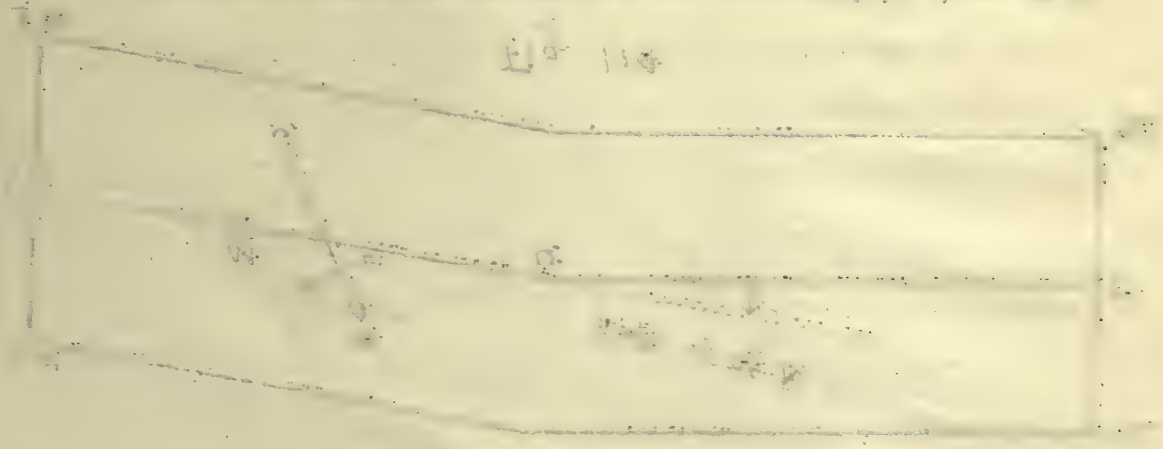


Fig. 114

The place where the prospecting was conducted is shown at M; from here a line is run N60° 10'E to O where it is made to deflect 12° 05' R (or its bearing from this point is N 72° 15' E) thence to C'; the line is also run from M, S 60° 10' W to C, the total length of the center line COC' is 1500 feet; suppose the bearing of the end line SW to NW is N 20° 00' W, then the other end boundary must also bear N 20° 00' W, i.e., the end boundaries must be parallel; the sideboundary lines may be 300 feet (perpendicular distance) on each side of the center line COC'.

These lines and dimensions may be laid out by the prospector or claimant and the approximate location, and necessary witness marks, such as stakes, bearing trees, etc., described by the claimant in filing his claim; an official survey is not required for this purpose.

Subsequently before a patent can be secured, it is necessary that an accurate survey shall be made by a qualified Mineral Land



Surveyor, and it is upon his report that a patent is issued.

The official surveyor is not rigidly bound by the claimant's survey, but he aims to follow consistently the lines and points previously established when these conform to good practice and do not interfere with the rights of other claimants or established surveys.

The survey must show the relation to any existing U. S. Land Survey in the region where situated. This is usually done by giving the true bearings of the boundary lines of the claim, and at least one corner of the claim is tied to a section or quarter section corner where possible.

The Deputy Mineral Surveyor is a regularly appointed official and is under bond with severe penalties ~~for~~ to insure the faithful performance of his duties.

REFERENCES:

Breed and Hosmer, Vol. 1, pp. 321-371.

Johnson - - - - - - - - - Chap. XI

Raymond - - - - - - - - - Chap. XII

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PROBLEMS:

1. A transit with top-telescope was used to set a point down an inclined shaft; the angle read on the verticle circle was 58° 18'. If the slope distance from the horizontal axis was 312.5 feet, and the eccentricity of the auxiliary telescope was 0.33 feet, (a) what was the correction angle, and what the true angle of depression? (b) what was the horizontal distance? (c) what was the vertical distance?

2. It is required to measure the angle ACB with a transit equipped with a side telescope, set up over the point C; sights were taken on A and B with side telescope and the angle read on the vernier plate was 61° 39'; if the distance from C to A was 212.3 feet, and from C to B 187.6 feet, and the eccentricity of the auxiliary telescope was 0.31 feet, what were the correction angles and the true interior angle ACB?

3. A mining claim is 1500 feet along the center line, and includes 300 feet parallel thereto on either side, and is further described as follows: The place of discovery, M, is 450 feet S 80° 00' E of the west boundary of said claim which boundary is made at right angles to the center line of the claim; 600 feet east of the west boundary the center line deflects left 23° 15'. Lay out the claim on a scale one inch equals 300 feet, drawing the bounding lines, the center line, and showing the bearing and distance of each line.

UNIVERSITY OF CALIFORNIA EXTENSION DIVISION

Correspondence Courses

Surveying-1B Elements of Surveying Mr. Swafford

Assignment 37

Hydrographic Surveying

Forward: The purpose in this assignment is to give some of the methods in that special branch of topographic surveying known as "Hydrographic Surveying". We will not presume to enter upon an elaborate treatment of the subject, but to give rather the problems and methods met with in the ordinary work of surveying. Only those problems which employ the usual tools of the surveyor will be discussed, with perhaps the addition of a few of the simpler but indespensable instruments required for such work.

(268) Scope of the Subject:

Hydrographic Surveying, although properly classed as topographic surveying, is more particularly regarded as a highly specialized branch of the subject. In all of its problems it deals with water, water surfaces, depths, streams, etc. Hence any survey that has to do with seas, lakes, harbors, rivers, and other aqueous bodies is classed as hydrographic, and the surveyor is here confronted with problems calling for treatment peculiar to each case, but concerned always with water, either in quiescent form or flowing. The intimate relation to land topography will generally be apparent.

(269) Problems of the Subject

A few of the problems met with in hydrographic surveying are:

(a) The measurement and mapping of the coast lines of lakes, ponds, and seas; (b) the determination of the forms of sub-aqueous areas covered by them; (c) the courses and depth of channels, presence and location of submerged rocks, reefs, and shoals; (d) the rise and fall of tides; (e) and the flow of streams, their slope, nature of channel, change of channel, velocity of flow, and the quantity of discharge and its nature.

The scope of the subject---its methods and problems---are indeed of prodigious importance and often call for highly specialized treatment. They must therefore not be considered of trifling nature suggested by this passing view of elementary phases of the work; the student should realize that very often works and structures of immense importance and involving intricate engineering undertakings are dependent upon the survey conducted in many cases.

To enumerate again: The many kinds of engineering ventures depending upon a hydrographic survey are: the improvement of harbors, erection of lighthouses, building of sea-walls, dredging of rivers, building of bridges, dams, tunnels, drainage of areas, and numerous other works.

(270) Survey of Shore-line:

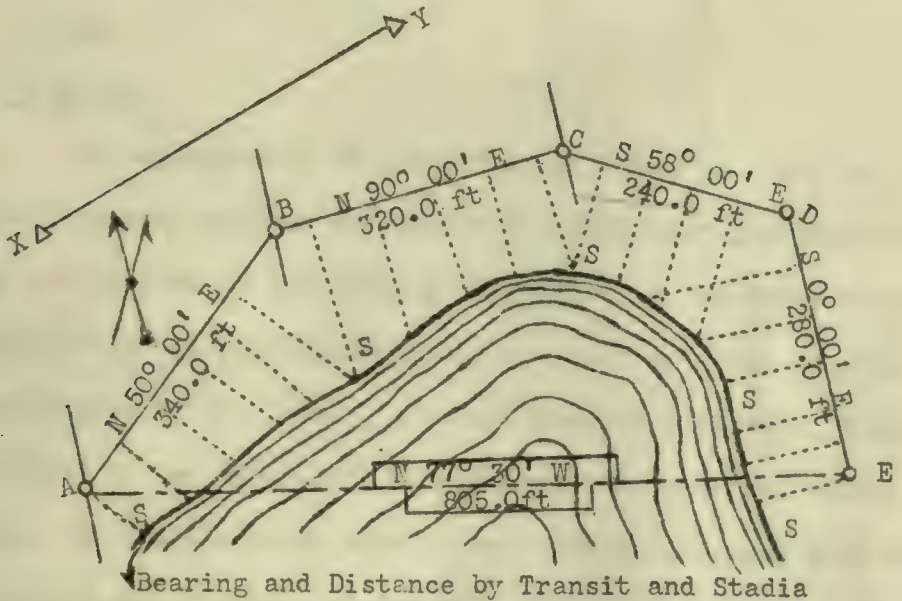
When it is desired to survey the shore-line of a lake or bay, for example, and this for the purpose of determining its boundary or for inclusion upon a map, it is the practice to run a traverse at a convenient distance from the shore-line upon the land, and to then establish the requisite number of points for defining the low-water

shore of the body of water. This traverse is generally of the open type, but even in such a case, though the closing course of the traverse cannot be measured directly, it is often desirable and expedient to secure a closing line by stadia or by triangulation, since such line would afford a valuable check on other elements of the survey.

If, as often happens, the coast-survey is an integral part of other topographic features, which have been based upon a triangulation system, then in turn a suitable net is projected to furnish the required basis of the hydrographic portion; or a special base, in some instances, is carefully established and the triangulation then proceeds from this base by the usual methods to include the hydrographic features sought.

If a traverse has been run, off-sets are then taken from designated points on the traverse lines to the points on shore at as frequent intervals as the character of the coast may require. These points may be measured by tape where great accuracy is required and the nature of the ground may permit, or by stadia when a lesser degree of accuracy is allowable or the intervening terrain renders direct measurement inconvenient or impossible;---this might be necessary on account of intervening marshy stretches, or where precipitous rocks stand in the way of chaining. Triangulation must sometimes be resorted to.

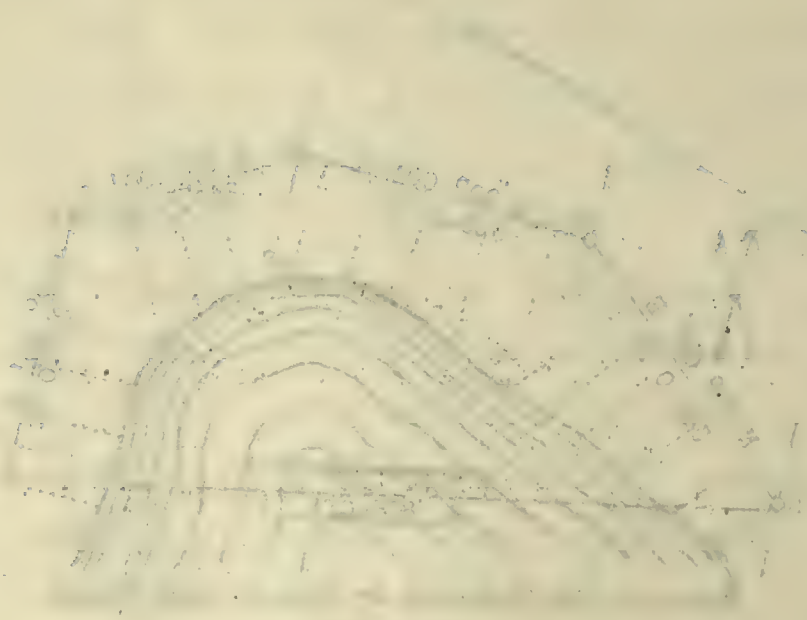
The illustration, Fig.115, will show the combined methods along a coast-line, consisting of a portion of a bay. Here the



Bearing and Distance by Transit and Stadia

Fig. 115

transit points A to E are located at convenient distance from the shore-line (exaggerated in the figure) and perpendicular off-sets are shown by dashed lines; through the points thus determined the low-water line SSSSS is drawn. The line AE, shown by a broken line, is conveniently found by stadia, and its bearing and distance also computed by latitudes and departures, as a check upon other elements of the traverse. The points A, B, C would conveniently be located by triangulation from X and Y, two triangulation stations of the land topography. The bearing and distance of each course of the traverse is found by transit and tape, or by transit and stadia. Besides the careful sketch shown here (drawn approximately to scale), a full set of notes in tabular form is kept and the distances along the transit lines to off-set points as also the lengths of off-sets



are also tabulated. From such data a faithful map of the coast is drawn.

(271) Soundings.

The measurements of the depth of bodies of water are called soundings and are made at specified points over the water surface for the purpose of determining the nature of the sub-aqueous area. The work is usually carried on from a boat by means of poles or lead-lines; if the former, a stiff narrow rod of wood, generally weighted at the lower end, and graduated in feet and tenths is used. In shallow water rods up to 15 feet in length graduated as described may be used to advantage, but longer rods are difficult to handle, and even with the shorter rods much decterity is required, especially in water in motion from currents of tidal flow.

For depths greater than two fathoms (1 fathom = 6 feet) lead-lines are employed. These consist of hemp cords, 3/8" to 1/2" size, carrying a lead weight at the lower end, weighing 5 to 20 pounds. The cord is suitably marked by tags at each foot and half-foot interval, after being wet and stretched, so that the shrinkage caused by wetting is in a measure eliminated. As this precaution is only temporary in character, and is likely to be affected by long use, the line and its graduations should be tested from time to time by comparing with a standard length, as a tape or the distance between two fixed points on shore.

Leads are of several forms depending upon the purpose; one form has a cup-like cavity at the lower end, which is designed to

lift a small quantity of the bottom material, from which the character of the sub-area may be judged---this may be mud, gravel, sandy sludge; or, in the absence of "sample", it would indicate rocky formation. When samples are especially desired, the cup-like cavity is smeared with tallow (called greasing the lead) to which the bed materials will more readily cling. A more elaborate device has a cup suspended at the bottom of the lead by a short stiff bar and a leather cap sliding upon this bar, acts as a cover to prevent the sample from washing out as the lead is raised.

As the sounding proceeds it is necessary to locate the point of each sounding. This may be done in various ways. The position of the boat may be measured "off-shore " by observing the angle by means of the sextant and afterwards computing the distance from known signals on shore. For this purpose it would be necessary to have four men, or more depending upon the kind of boat. Supposing that the boat is propelled by a single oarsman then the crew should consist of a leads-man, a recorder, an instrument man; the recorder may also serve the tiller, if the boat is thus equipped.

The leads-man is stationed in the bow and the man handling the sextant is immediately behind him, the oarsman occupies the middle seat and the recorder sits astern.

Another method requires the reading of angles from stations ashore. This is done by two transit-men with instruments set up at known points that are intervisible and that command a clear view of the water area over which the soundings are to be taken. This is in

reality the method of triangulation so common in topographic work generally. Fig. 116 shows points M,N,O,P connected by triangulation to a station T, the azimuth and distance of which are carefully determined. As the soundings A---B---C are in progress the transits are set up first at M and N, each sighting first on each other; then, at the instant a sounding is taken both sight upon the leadsman's station simultaneously; the moment of taking the sounding can be signaled from the boat by means of a flag. The transit-men then move to N and O, and again taking stations O and P. This is a very accurate and satisfactory method, but of course involves considerable equipment and two transit-men which, of course, means expense.

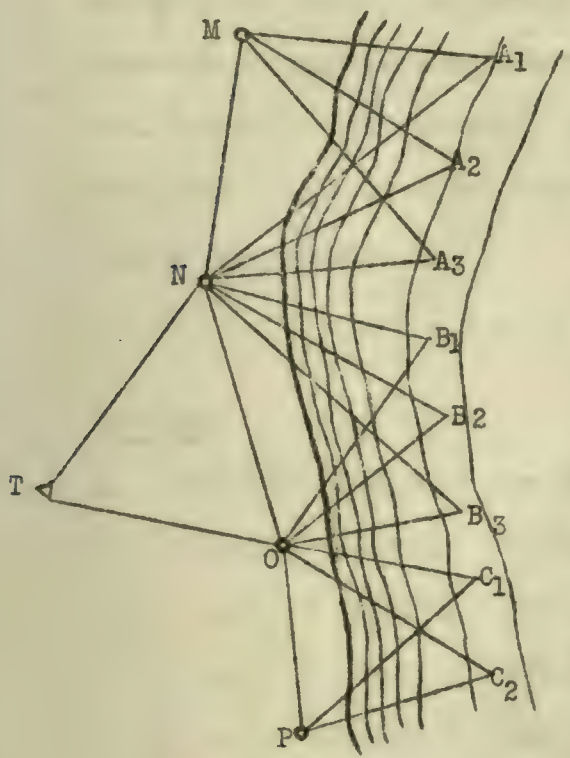
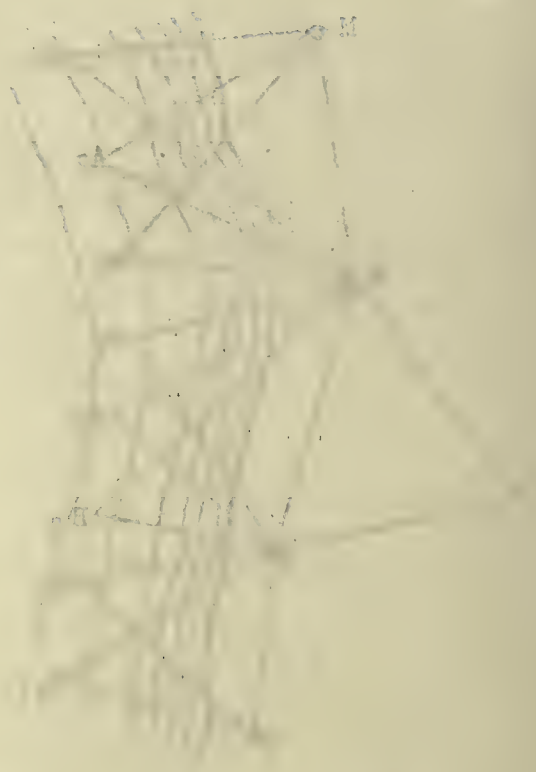


Fig. 116

If the soundings can be taken in range, which is expedient when a profile of the bottom of a stream, or bay or harbor is required, then the method requiring no transit is feasible. This is done as follows: Two posts are planted in range with the desired course of soundings; the posts are erected so that they may always be kept in view from the boat from which soundings are taken. It is well to have these signals painted white or some other bright color that is in contrast with the back-ground against which they may be viewed from the boat.



The sounding party in the boat keep these signals in range rowing in as nearly a uniform rate as practicable, and taking soundings at regular time intervals; thus the depths at approximately equal intervals are obtained. The profile may now be plotted in this particular range. If other profiles are desired, the range may be altered accordingly.

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Problems.

I. The following data are those shown in Fig. 115, page 4.

Course	Bearing	Distance
A-B	N 50°00' E	340.0 ft.
B-C	N 90°00' E	320.0
C-D	S 58°00' E	240.0
D-E	S 0°00' E	280.0

Required, the bearing and distance of E-A. If A and E are intervisible, how may the bearing and distance be measured directly?

II. Referring to Fig. 116, page 7, the following data are given. TN = 730.0 ft., TO = 680.0 ft., Angle NTO = 63°00', Angle NOB_2 = 75°00', Angle ONB_2 = 40°00'.

Required the distances NO, OB_2, NB_2.

UNIVERSITY OF CALIFORNIA EXTENSION DIVISION

Correspondence Courses

Surveying-1B Elements of Surveying Mr. Swafford

Assignment 38

Mapping and Office Work - Computing.

Foreword:

The work in this assignment will consist of mapping problems: 1st, A Profile of River Crossing; 2d, three traverses, each by a different method, all to be placed upon one sheet; 3d, A Topographic Map, in which the data given and worked out in detail in Assignment 33 will be made available. You are specially urged to make this a particularly profitable set of exercises, and to follow the directions given to the least detail. These exercises will serve as an index of your attainment in the work of the course, and will further display your aptitude for carrying out projected work and your mastery of the important details of office work in connection with field data.

Consult all references available and such other sources of information on drawing, mapping, and lettering as may be likely to contribute to your successful completion of the projected maps.

In order to prevent the maps from becoming folded or damaged in the mails, send them to the Extension Division in a substantial mailing tube.

Material:

For the three mapping problems the following material is required which may be purchased from the Keuffel and Esser Co.,

30 Second St., San Francisco, or from the A Lietz Co., 61 Post St., San Francisco.

	Approximate cost.
1 yard Plate A Profile Paper 20" wide.	$0.25
2 yards Detail Paper 36" wide.	.40
1 yard Tracing cloth 36" wide.	.45
Total	$1.10

This does not allow for enough extra to use in case one sheet is damaged, so proper care should be exercised to prevent mistakes in inking, etc.

Mapping Problem 1.

Profile.

For this problem use 2 ft. of 10 in. Plate A profile paper. Plot the profile of the stream crossing given in the notes below. Horizontal scale 1" = 20', vertical scale 1" = 4'. Place station 0 so that the profile is well spaced on the sheet. Write the elevations on the horizontal lines. At each alternate heavy vertical line write the station number just below the lowest horizontal line.

Profile of Lorenzo River Crossing.

Sta.	Elev.	Remarks	Sta.	Elev.	Remarks
0+00	158.6		2+10	125.6	
+10	157.5		+15	126.8	
+15	157.2		+25	129.7	
+22	156.8		+30	130.5	
+30	155.2		+35	130.6	
+40	154.2		+40	130.3	
+48	152.8		+50	129.0	
+59	150.0	high water 1907	+60	127.4	

Sta.	Elev.	Remarks	Sta.	Elev.	Remarks
+75	148.3		+64	126.6	
+90	147.0		+76	126.8	
+95	147.1		+90	129.0	
1+00	146.8		3+00	130.3	
+05	146.6		+08	131.0	
+10	146.3		+20	131.6	
+20	146.0		+35	133.0	
+24	145.7		+50	134.8	
+32	145.2		+60	135.8	
+36	144.2		+75	137.8	
+48	140.4	present water level	+80	140.4	present water level
+70	133.0		+90	148.1	
+82	130.2		4+00	151.8	
+86	128.6		+10	153.9	
+96	124.9		+20	155.5	
2+00	124.8				

Plot the profile lightly in pencil (profile paper will stand very little erasing). In plotting each point place the pencil point on the plotted position and draw back to the preceding point a free hand straight line. Smooth out this broken line when the profile is inked.

Before inking you should check the profile carefully and send it to the Extension Division for the approval of your instructor. For inking use a contour pen or a ball pointed pen, making the line about the same width as the heaviest lines of the profile paper. Ink the profile in black. Place the following standard form of title in the upper middle part of the drawing:

University of California
Extension Division

Plane Surveying Assignment 38

Profile of Lorenzo River Crossing

(Name) (Date)

Scales: 1 in. - 20 ft. horizontal
1 in. - 4 ft. vertical

Great care and much taste must be exercised in lettering, especially in titles. Use guide lines and slope lines and a simple letter of uniform style.

Mapping Problem 2.

Traverses.

General Instructions. Use detail paper 20" x 27" with standard border as indicated on page 8 of this assignment. Make the finished sheets 18" x 24" with border 3/4" inside those dimensions. Considering the north-south line to be parallel to the shorter side of the drawing, let the origin of plotting coordinates be at the lower left (SW) corner of the border line.

Complete the entire drawing in pencil and return it to the Extension Division for the approval of your instructor. Do not do any inking of the drawing until it has been approved in pencil form. Check it over very carefully before submitting it.

There are three traverses to be plotted on the one sheet. The scale for each traverse is 1" = 200'. Show directions and length of every course in each traverse as in Blueprint Problem 3, Assignment 6, (or Figure 199, Breed and Hosmer). Use standard north point illustrated at the end of this assignment and the form of title already indicated. (C.E.Traverses - Chord, Tangent and Protractor Methods). Place the title in the lower right hand corner of the plate. Use soft pencil for lettering so that erased mistakes will not show by reason of indentation of the paper.

TRAVERSE "A"

Traverse "A" will be plotted by deflection angles, using the tangent method. The deflection angles will be computed from the notes given below. Reference: Assignment 30, p. 3; Breed & Hosmer; Arts 490-491. Show in this traverse plot a typical construction for a deflection angle less than 45°, and one for a deflection angle between 75° and 90°.

Sta.	Deflection	Bearing	Distance (ft.)
A_1	( )		
		N 6° 37' W	932.4
A_2	( )		
		N16 43 E	500.0
A_3	( )		
		N 2 38 E	1018.0
A_4	( )		
		N78 53 E	664.9
A_5	( )		
		S25 57 E	900.6
A_6	( )		
		S21 28 W	468.1
A_7	( )		
		S10 22 E	938.7
A_8	( )		
		S45 23 W	579.4
A_9	( )		
		N87 57 W	715.3
A_1	( )		

Note: Co-ordinates Sta. A_1 (270.0 N, 340.0 E)
Check deflection angle and bearing closure.

TRAVERSE "B"

Traverse "B" will be plotted by laying out the bearing angle of each course by the chord method, drawing a north and south construction line through each transit point. The bearings are to be computed from the following notes. Reference: Assignment 30, p. 5;

Check orientation angle and marking element.

Arts. 492-494. Show in this traverse a typical construction for a bearing angle less than 45° and one for a bearing angle between 75° and 90°.

Sta.	Deflection	Bearing	Distance(ft.)
B_1	-41° 30' R		
		N 26° 38' W	1171.3
B_1	-72 15 R		
		( )	722.9
B_3	-59 25 L		
		( )	756.4
B_4	-100 00 R		
		( )	1065.0
B_5	-66 6.5 R		
		( )	679.0
B_6	-50 28.5 R		
		( )	1015.0
B_7	-27 10 L		
		( )	1180.0
B_8	-116 15 R		
		( )	955.2
B_1			

Note: Co-ordinates of Sta. B_1 (430.0 N, 1925.0 E).
Check deflection angle closure and bearing closure.

TRAVERSE "C"

Traverse "C" will be plotted with a 14" paper protractor, using azimuths from the south point which will be computed from the following notes. Reference: Assignment 30, pp. 6-8; Breed & Hosmer; Arts. 481-483.

Sta.	Deflection	Azimuth	Distance (ft.)
C_1	73° 35' R		
		159° 10'	635.0
C_2	32 15 R		
		( )	455.1
C_3	9 15 L		
		( )	410.3

Sta.	Deflection	Azimuth	Distance (ft.)
C_4	-61° 40' R		
		( )	-468.0
C_5	-124 29.5 L		
		( )	-516.9
C_6	-126 4.5 R		
		( )	-460.2
C_7	-32 50 R		
		( )	-870.8
C_8	-91 45 R		
		( )	-918.1
C_9	-29 15 L		
		( )	-596.0
C_{10}	-34 40 R		
		( )	-450.1
C_{11}	-70 10 R		
		( )	1048.0
C_1			

Note: Co-ordinates of Sta. C_1 (720.0 N, 3300.0 E)
Check deflection and azimuth closure.

Mapping Problem No. 3.

Topographic Mapping Problem

Traverse. The field notes for the traverse to be used in connection with this problem have already been included in Assignment 33. In this assignment you were required to compute horizontal distances, differences in elevation, and elevations; and to balance the traverse, adjusting the elevations and distributing any closing difference in proportion to the lengths of the respective sides.

Map. Using the traverse computations indicated plot the traverse by the co-ordinate method, checking by scaling the lengths of the sides. Write in pencil on the map the length and bearing of each

Item	Definition	Amount	Distance (ft.)
[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]

Table 1: Dependencies of Item [illegible] (1754 Rs. 2000.00 %)

Meeting Problem Use [illegible]

Subcontracts Working Problem

Directions The final notes for the chairmen by be used in
[illegible] with this problem. Some already been developed in earlier
[illegible] We will [illegible] to complete Department [illegible]
[illegible] the [illegible] adjustment the [illegible] and [illegible]
[illegible] [illegible] [illegible] [illegible] [illegible]

side. The map is to be plotted on a scale of 1" = 60', contour interval 5 ft., on detail paper of good quality, outside dimensions 18" x 24". Use 3/4" margin and standard border. Assume the true north parallel to the shorter edge of the border, and be careful to select the position of the origin of co-ordinates in such a way as to center the drawing upon the sheet. The title is to be placed in the lower right hand corner of the sheet, and is to be of standard form. The map is to be completed in pencil only and submitted to the instructor for approval. Do not erase correct construction work on the pencil drawing, either before or after approval. Use a 4H, or harder pencil for plotting traverse and side-shot points and a softer pencil for drawing contours and for lettering.

A tracing in ink upon tracing cloth (using the dull side) is then to be made of the approved map. Tracing and original are then to be submitted to the instructor, together with computations and tabulations. For details consult references listed below and those given in Assignment 33.

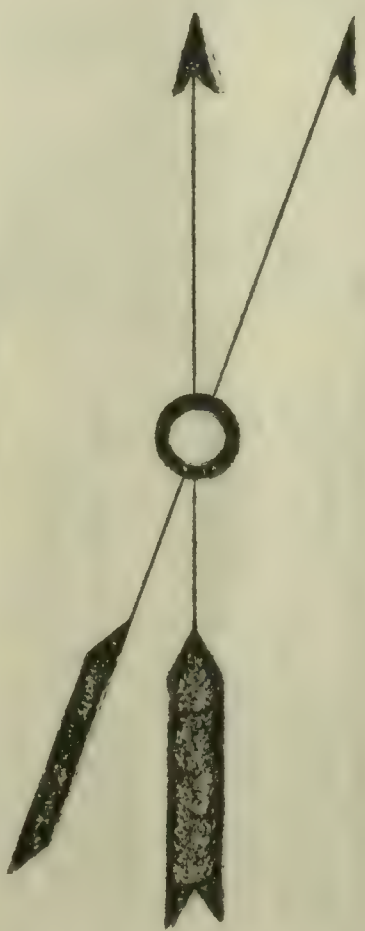
References:

Breed & Hosmer Vol. 1, Chap. XI and XVII; Vol. II, Chap. XI.

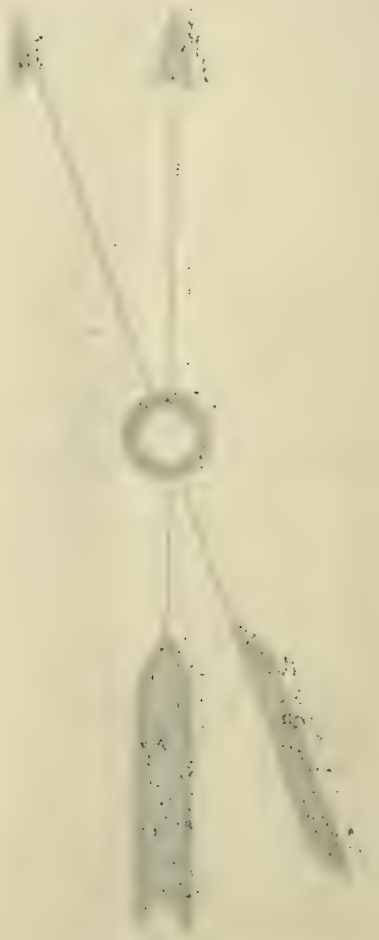
Tracy Chap. XI.

Raymond Chap. XII, also plates 1-3, 7.

Note: Bear in mind that the originals of all three mapping problems are first to be submitted to the Extension Division in pencil only for the approval of your instructor. They will then be returned for inking-in and should be re-submitted in final form for the proper grade.



Standard Border and North Point

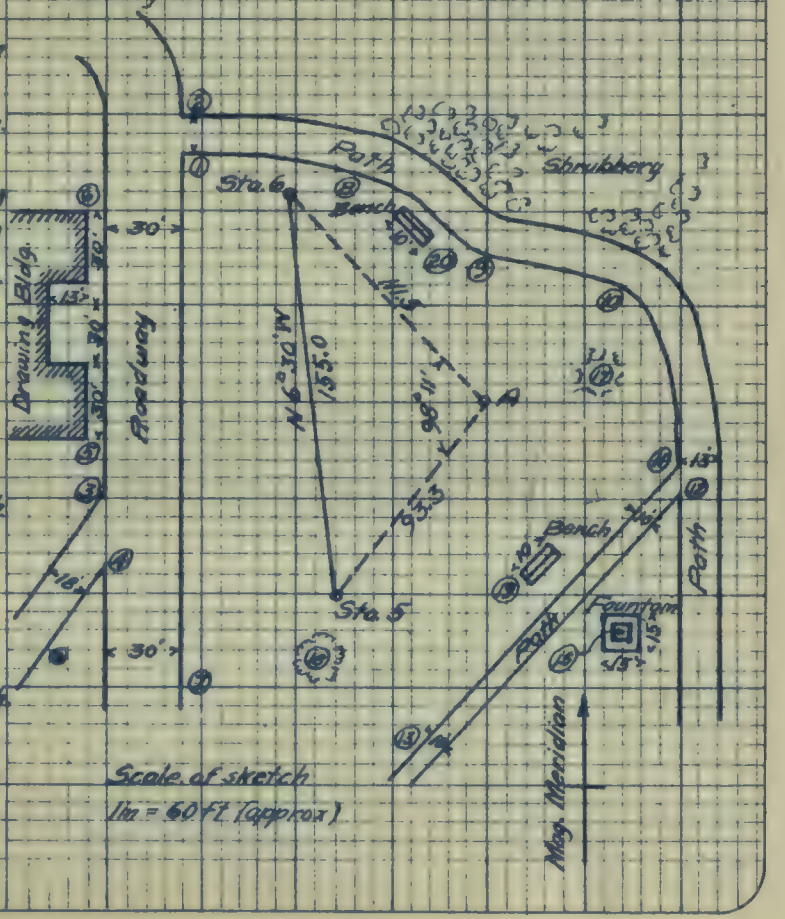


PROB. NO. 8. DETAILS WITH TRANSIT AND TAPE.

Station	Object	Angle	Distance	Remarks
				Used line 5-6 of traverse as base line of survey.
				Length of 5-6 = 155.0 feet.
				Magnetic Bearing = N 6° 30' W.
A	Sta. 6	0°00'	111.5	All angles and distances taken
	Sta. 5	98°11' L	93.3	from line A-6 and Sta. A.
	1	7°41' L	150.8	Int. edge of path & roadway.
	2	3°55' L	160.3	" " " " " "
	3	59°30' L	159.5	" " " roadways. (N.W.)
	4	71°00' L	160.0	" " " " (S.W.)
	5	52°13' L	154.4	S.E. cor. S.E. wing Drawing Bldg.
	6	21°00' L	170.6	N.E. " N.E. " " "
	7	89°30' L	159.1	E. edge of roadway.
	8	12°45' R	102.3	S. edge of winding path.
	9	41°35' R	58.4	" " " " "
	10	8°45' R	67.2	" " " " "
	11	150°26' R	73.5	N.W. int. of winding path & str. path.
	12	159°02' R	79.5	S.W. " " " " " "
	13	120°31' L	139.6	N.W. edge of straight path.
	14	169°18' L	129.7	W. " " winding path.
	15	166°39' L	100.5	Center of Fountain - 15'x15'.
	16	78°10' L	197.1	S.E. edge of roadway S. of Drawing Bldg.
	17	119°30' R	39.5	W. side of oak. Diam. = 3 feet.
	18	102°37' L	112.6	N. " elm. Diam. = 2½ feet.
	19	146°47' L	67.0	W. cor. Bench N.W. of Fountain
	20	22°34' R	65.5	S. " N. of A. → 10' long

Equipment:
Berger transit #6945.
100-foot steel tape #15.
2 Plumb-bobs.
2 Line rods.
Chaining Pins.
Reading Glass.

October 1, 1907.
Overcast & swithy.
Hemming, C. J. & Chain
Carter, H. D. } Rod.
Michaels, L. V. }



UNIVERSITY OF CALIFORNIA EXTENSION DIVISION

Correspondence Courses

Surveying-1B. Elements of Surveying Mr. Swafford

Solutions to Assignment 38.

Computations for Mapping Problem 2.

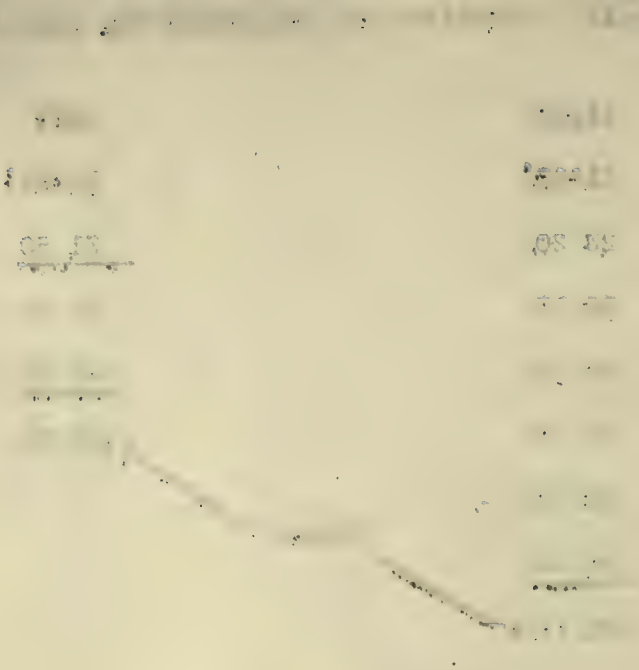
Sta.	Deflection	Bearing	Tangents	Distance(ft.)	Cotan
A_1	(81°20' R)		6.5606		0.1524
		N 6° 37' W	0.1160	932.4	
A_2	(23°20'R)		0.4314		
		N16 43 E		500.0	
A_3	(14°05'L)		0.2509		
		N 2 38 E		1018.0	
A_4	(76°15'R)		4.0867		0.2447
		N78 53 E		634.9	
A_5	(75°10'R)		3.7760		0.2648
		S25 57 E		900.6	
A_6	(47°25'R)		1.0881		
		S21 28 W		468.1	
A_7	(31°50'L)		0.6208		
		S10 22 E		938.7	
A_8	(55°45'R)		1.4687		0.6809
		S45 23 W		579.4	
A_9	(46°40'R)		1.0599		
		N87 57 W		715.3	
A_1	(81°20'R)		6.5606		0.1524

Note: Co-ordinates Sta. A_1 (270.0 N, 340.0 E)
Check deflection angle and bearing closure.

Right	Left
81°20'	14°05'
23 20	31 50
76 15	45 55
75 10	360 00
47 25	405 55
55 45	
46 40	
405 55	

Check

Sta.	Deflection	Bearing	Chords	Distance(ft.)	Sin $\frac{\alpha}{2}$
B_1	41° 30' R				
		N 26° 38' W	0.4606	1171.3	0.2303
B_2	72 15 R				
		(N 45 37 E)	0.7752	722.9	0.3876
B_3	59 25 L				
		(N 13 48 W)	0.2426	756.4	0.1213
B_4	100 00 R				
		(N 86 12 E)	2.3666	1065.0	0.6833
B_5	66 6.5 R				
		(S 27 41½ E)	0.4784	679.0	0.2392
B_6	50 28.5 R				
		(S 22 47 W)	0.3870	1015	0.1935
B_7	27 10 L				
		(S 4 23 E)	0.0766	1180.0	0.0383
B_8	116 15 R				
		(N 68 08 W)	1.8562	955.2	0.9281
D_1		N 26 38 W	0.4606		0.2303



Note: Co-ordinates of Sta. B_1 (430.0 N, 1925.0 E)
Check deflection angle closure and bearing closure.

Right	Left
41°30'	59°25'
72 15	27 10
100 00	86 35
86 06½	360 00
50 28½	446 35
116 15	Check
446 35	

Sta.	Deflection	Azimuth	Distance (ft.)	Bearings
C_1	73° 35' R			
		159°10'	635.0	N 20°50' W
C_2	32 15 R			
		(191 25)	455.1	N 11 25 E
C_3	9 15 L			
		(182 10)	410.3	N 2 10 E
C_4	61 40 R			
		(243 50)	468.0	N 63 50 E
C_5	124 29.5 L			
		(119 20½)	516.9	N 60 39½ E
C_6	126 4.5 R			
		(245 25)	430.2	N 65 25 E
C_7	32 50 R			
		(278 15)	870.8	S 81 45 E

Sta.	Deflection	Azimuth	Distance (ft.)	Bearings
C_8				
	91° 45' R			
		(10°00)	918.1	S 10°00'W
C_9				
	29 15 L			
		(340 45)	596.0	S 19 15 E
C_{10}				
	34 40 R			
		(15 25)	450.1	S 15 25 W
C_{11}				
	70 10 R			
		(85 35)	1048.0	S 85 35 W
C_1		159°10'		

Note: Co-ordinates of Sta. C_1 (720.0 N, 3300.0 E)
Check deflection and azimuth closure.

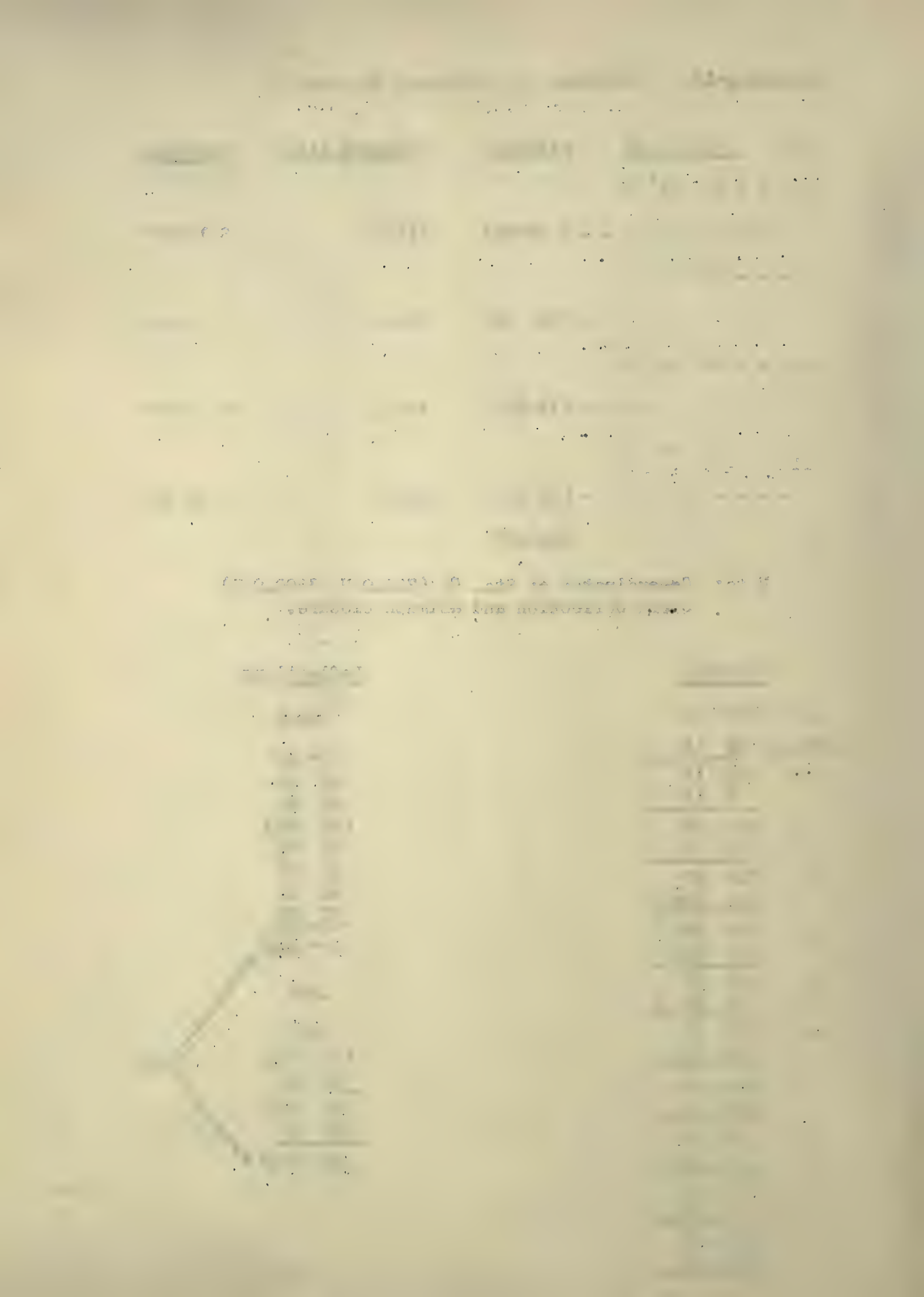
Azimuths

Az C_1	159° 10'	
Defl. C_2	32 15	R
C_2	191 25	
	9 15	L
C_3	182 10	
	61 40	R
C_4	243 50	
	124 29½	L
C_5	119 20½	
	126 04½	R
C_6	245 25	
	32 50	R
C_7	278 15	
	91 45	R
	370 00	
	360 0	
C_8	10° 00'	
	29 15	L
C_9	240 45	
	34 40	R
	375 25	
	360 00	

Deflections

Right		
	73° 35'	
	32 15	
	61 40	
	126 04½	
	32 50	
	91 45	
	34 40	
	70 10	
	522° 59½'	
Left		
	9° 15'	
	124 29½	
	29 15	
	162 59½	
	360 00	
	522° 59½'	

Check



Azimuths, con.

C_{10}	15°	25'		
	70	10	R	
C_{11}	85	35		
	73	35	R	
C_1	159°	10'		Check

Mapping Problem No. 3.

For computations of Mapping Problem No. 3, see those furnished with Assignment 33.

Included with Assignment 38 are the following maps: Profile Map, Traverse Map and Computations, Topographic Maps (pencil original and tracing) and computations.

UNIVERSITY OF CALIFORNIA EXTENSION DIVISION

Correspondence Courses

Surveying-1B Elements of Surveying Mr. Swafford

Assignment 39

Rights, Duties, and Privileges of the Surveyor.

Foreword:

The title of this assignment implies the fact that the surveyor in the pursuit of his work is vested with certain rights peculiar to his profession and that he is accorded privileges that might be wanting to one not engaged in such work as he may be called upon to do; also that he is obligated by the quasi-official character of his calling to perform certain functions in a way and to an extent perhaps not required of others. There are certain delicate and difficult relations between the surveyor and his client, things he owes to the latter and to himself. There are some obligations to the employer or the public of which he must be conscious while performing his duties, and he must endeavor always to place himself four-square with his work. Let us quote in this connection from the introduction to the course, Assignment 1, Page 6:

To properly accomplish the work of his chosen profession, "The surveyor must be a man at once honest, sincere, dependable, energetic, ingenious, and observant. He must be patient in his work and with other people. He must be ready to give an unbiased opinion as to the rights of disputants when called upon to do so, and should always be sure of the correctness of his work or its

limitations before submitting results to his employer or to others concerned. He must have a thorough knowledge of the fundamental principles of surveying such as will enable him to solve all his problems correctly."

It will hence be the purpose in this assignment to follow the outline as suggested by the foregoing.

The surveyor must be just in his decisions, and he must observe a balancing of claims by rival parties. Fairness should always be the governing motive in all disputes, since the surveyor's work is often for the purpose of settling controverted claims.

The mere fact that one has the knowledge or skill that makes him a surveyor does not vest him with authority or judicial functions. His training ought rather to give him a fair understanding of the limitations that beset the practice of his profession, and lead him to a moderate view of his importance. His functions, apart from the facts that his labors in any case have discovered, are advisory; therefore he should be a good councillor rather than a judge. Men will listen to his counsels, but few would accept his decrees.

The surveyor must also distinguish carefully what privileges are his, what rights he may claim, and what duties he must perform. He is a servant, an agent, an officer in some cases with duties fully set forth by statute; he must be punctilious in the performance of these duties, but must at the same time avoid assuming those that are distinctly not his; he must modestly attend to his business--his own and not that of others.

In other words he must not trespass. This is a strong word, but trespassers are sometimes punished for their offenses when no offense was intended, and the surveyor is peculiarly placed in many instances while endeavoring to perform his duty.

A surveyor may pass through or over lands of another in making measurements, but he must not commit any nuisance or destroy any property, such as crops, trees, or buildings; in case any infringment of property rights is made the surveyor is liable for trespass and may be punished therefor.

It is often convenient to remove trees, brush, etc. from a line of road or other survey; but this can be done only by permission or by securing a right of way. Growing or standing crops of any sort come under the same rules.

A surveyor should, in the absence of permission to cross any property where damage by himself or assistants might result, devise some method of determining distance or bearing of line or other data by which it is unnecessary to enter the property.

Railroads and private roads, canals, etc., usually secure right of way, or at least an option promise carrying with it the proper permit to enter the property for the purposes of surveying. In such cases the duty (morally) of the surveyor is evidently to do as little damage as possible to crops, timber, and other property.

In the public land surveys, over public property, the surveyor may exercise a wider liberty, but this should never be construed

to mean wanton destruction of timber or crops. When in a public land survey, or that of a highway, it is necessary to enter or cross private lands, the surveyor is justified only so far as is absolutely essential in removing or destroying property, but the employing principal, the state or county, or municipality may be required to satisfy any just claims for damage committed by such work. Here also the surveyor is to avoid wanton waste or damage.

The surveyor's duty respecting all surveys conducted by him is to do all work with the full purpose of securing ample data, the making of suitable sketches for his own guidance and that of others who may come to use the material gathered by him.

Notes must be full, clear and complete; they must not present any ambiguity , or otherwise doubtful matter; figures must be clear and plain so as to afford a ready interpretation; the data recorded in the notes must show no alterations or erasures. If perchance incorrect quantities are entered in the notes and the necessary alterations are to be made, it should be done by crossing out the erroneous entry and writing above (or below) the correct quantities. In case a sketch is made to accompany tabulated quantities, the lengths (distances), bearings (angles), and other related parts, such as names of owners of adjoining property, cardinal directions, etc., must be indicated in proper place and in suitable form on the sketch, thus connecting the sketch with the tabulated data which it is desired to illustrate and clarify.

a screen is made to accompany keyboarded queries, the Jorge

Throughout this course your attention has repeatedly been called to the forms of notes approved for various cases; other forms are made use of, and in some cases a different form may recommend itself, but a careful consideration of form is a matter of importance in all cases. Remember that notes of surveys are generally made for others than the surveyor himself to use--to read and readily and clearly understnad; hence, any departure from approved form, any omission of important data, or the absence of sketch and explanatory notes may lead to much confusion, and the work of the surveyor may be rendered worthless through faulty or insufficient notes.

A practice of some surveyors of purposely suppressing important data, or necessary directions or explanations in order that they may monopolize a survey when made, is to be deprecated; this is a species of dishonesty seldom met with, but instances are common enough to warn the novice to avoid any practice so flagrantly unfair--unprofessional. This brings us to consider an important matter, which is the ownership of surveys.

If the surveyor has acted in the capacity of an employee, servant, or agent, i.e. has done the work for hire and has received compensation for his services, evidently the product of his labors connected therewith belong to the employer. Hence the right to monopolize the results--the notes, data, maps, or information gathered for the purpose belong to the one who pays for the survey. If such a one is a private individual, to him; if it is for a company,

firm, or corporation the rights of ownership are vested in the company, firm, or corporation. Under this last classification may come a municipality, county , or state, and in such case the survey becomes a public document and should be made a matter of record.

In making a survey the following things should be carefully attended to:

1. The bearings of all lines; preferably the true bearings; but if the magnetic bearings are given instead, then the magnetic declination should also be correctly shown or stated.

2. The lengths of all boundaries, offsets, and tie lines.

3. The designated name or number of the tract, block, or lot; if the land is part of a subdivision or additon to a town, this fact should be stated and the official record of such subdivision or addition clearly referred to.

4. In cities the names of streets; also bodies of water bounding same, and names of contiguous owners.

5. The angles of lines of adjacent lands with names of owners.

6. A plain title, a scale, and a meridian arrow; also the date of the survey and the signature of the surveyor with his official title, if any.

7. If required, the recording of the survey with all parts complete-- notes, map,etc., in the office of a county recorder, with a municipal, state, or government official as required.

There is nothing wrong in a surveyor's use of information of such character in the further prosecution of the work of his profession, but he has no rights of monopoly of such information.

He has, when work is done as above detailed, done only his duty; if he has done this conscientiously and correctly his rewards are large and sure.

Problem to Accompany Assignment 39:

As a problem to accompany this assignment set up the notes of an ideal land survey giving the data, sketch, and explanatory notes in acceptable form. Follow the specifications given in the assignment from 1 to 6 inclusive. This means a small map on a sheet $8\frac{1}{2}$" x 11". The map should be carefully drawn in pencil first and finally inked. Do careful lettering using taste in all the work.

Submit this as evidence of your having profited by this course in Place Surveying.

UNIVERSITY OF CALIFORNIA EXTENSION DIVISION
Correspondence Courses

Surveying-1B Elements of Surveying Mr. Swafford

Assignment 40

Rights, Duties, and Judicial Functions of Surveyors.

Foreword. In this, the concluding assignment of this course in the Elements of Surveying, it has been deemed best to further call your attention to the fact that while as a surveyor you have certain rights due to your quasi-official position and while certain important and specific duties must compel you in an ~~undesirable~~ undesirable course, you are also hedged about by other things that limit the scope of your authority and especially your judicial functions.

(277) There is a tendency with those whose education and experience has been along mathematical and scientific lines to become imbued with a high regard for their own judgments and conclusions. A training along such lines, however, should render you modest and cautious; it should make plain the fallibility of all human agencies; it should teach the exercise of deference to the opinions of others and of patience and forbearance. Instead of reaching conclusions hurriedly and pronouncing judgments boldly a scientifically trained mind should take note of all phases, evidences, or testimonies, balance them carefully, and draw deductions only after full deliberation.

Again a hasty or faulty conclusion is sure to carry you

into error that must later be rectified by much labor and sacrifice. The consequences of false conclusions are often far-reaching and attended with confusion and loss to all concerned. They often provoke disagreement and expensive litigation.

(278) In Assignment 20, Art. 153 the subject of the location of lost and obliterated corners of the public land surveys was briefly treated; the surveyor who assumes to attempt such work should enter upon it only after fully understanding the method in each case and by following explicitly and carefully the full instructions afforded by the General Land Office. In the execution of your duties in such matters you must recognize that what was done and what was intended to be done when the original survey was made may be at variance; and that it was what <u>was done</u>, not what was <u>intended</u>, that established the corners and their connecting lines that stand as the tangible record; and that this record must not be changed for any reason, whatever the findings in any case. When you have to the best of your ability executed the commission intrusted to you, your own status has not been advanced beyond that of an expert and you should neither regard yourself nor permit others to believe that you have any judicial prerogative whatever. You may counsel or advise; you may use the weight of your knowledge and skill, or the

confidence reposed in your ability arising from long experience and conscientious service to advise or concilliate, but you must not act the judge.

(279) Thus it will be seen that the relation of the surveyor to his client is a delicate one; that he owes it to himself to be truthful, painstaking, honest; he must apply the knowledge and the arts of his craft to the discovery of the truth and he is then and only then justified in urging his conclusions and findings for acceptance by those he would serve.

The fallibility of man and the imperfections in the systems of surveys, even when the greatest precautions have been taken to establish and monument important surveys, have called for the intervention of the courts in myriads of cases and have called for some notable decisions which have a historic as well as a judicial bearing.

The surveyor is not called upon to know the law as expressed in these decisions, but it is expedient that he should know how these laws may affect the rights of his client, for he may be asked to advise or arbitrate, and then a knowledge of legal decisions and rulings would assist his judgment.

(280) "In making a resurvey where there is considerable uncertainty, as in the case of disputed boundaries, the surveyor has no official power to decide any disputed point. He can only act as an expert, and give an opinion as to what is the most

probable solution of the difficulties in question. If the interested parties do not agree to accept his decision, the question must be settled by the courts." (Quoted from Gillispie's Surveying.)

A code of rules for the guidance of surveyors is found in the above work on pages 346-351. The student of this course is advised to make himself familiar with them.

Another compilation of valuable instructions is to be found in a paper prepared for the Michigan Society of Surveyors and Engineers by Justice Cooley of the Michigan Supreme Court. A reprint of this paper is found in Raymond, and also in Johnson, to which the student is referred. This and the foregoing "rules" have become "classics" of the subject, "The Judicial Functions of the Surveyor." These then taken together with the court decisions above referred to constitute the "code" of the surveyor and should be relied upon for guidance in his work.

(281) A few quotations follow, taken from several sources, and designed to show the scope and limitations besetting the surveyor:

"No statute can confer upon a county surveyor the power to 'establish' corners, and thereby bind the parties concerned. Nor is this a question merely of conflict between state and federal law; it is a question of property right."

"In any case of disputed lines, unless the parties con-

cerned settle the controversy by agreement, the determination of it is necessarily a judicial act, and it must proceed upon evidence, and give full opportunity for a hearing."

"Nothing in what has been said can require a surveyor to conceal his own judgment, or to report the facts one way when he believes them to be another. He has no right to mislead, and he may rightfully express his opinion that an original monument was at one place, when at the same time he is satisfied that acquiescence has fixed the rights of parties as if it were at another. But he would do mischief if he were to attempt to "establish" monuments which he knew would tend to disturb settled rights; the farthest he has a right to go, as an officer of the law, is to express his opinion where the monument should be, at the same time that he imparts the information to those who employ him, and who might otherwise be misled, that the same authority that makes him an officer and entrusts him to make surveys, also allows parties to settle their own boundary lines, and considers acquiescence in a particular line or monument, for any considerable period, as strong, if not conclusive, evidence of such settlement.

(282) Reasonable persons are inclined to give weight to the findings of an expert, as the surveyor is supposed to be, and if he has also the authority of a public office to support his findings, they are likely to be convincing. But he must

not assume that his decisions are final as the courts, though competent testimony and other convincing evidence may modify or reverse the conclusions of the surveyor.

When, in case of disputed boundary the surveyor is called upon to render the service of locating monuments or running lines, after a complete finding as to the true facts, they should be submitted to the parties concerned with no attempt to urge upon either what would not be exacted of both. In other words the disputants should be led to see the facts as revealed in the survey and to regard these as of import to both. To secure a final settlement it is necessary that they agree to abide by the survey, and of course to make this binding the agreement should be reduced to writing and properly drawn and recorded. No parole agreement can secure the title as to property rights.

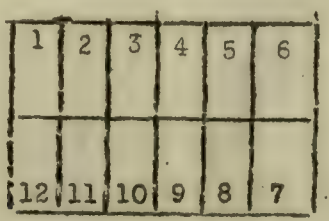
(283) In conclusion let us urge that you exercise both tact and modesty. You must not assume an arbitrary attitude nor permit yourself to mislead employing parties to believe that you are possessed of authority when you have none, or that your findings are final and inflexible. The courts are the only "last resort" of parties who disagree and disputants who cannot settle their differences otherwise must do so in the courts.

Questions and Problems

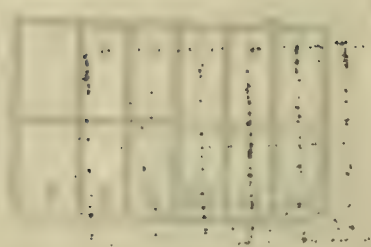
1. Show by diagram, fully detailed, how to proceed in laying out the south half of the N.W. quarter of section 15 in T6N, R4E, assuming that the two township corners on the south are located.

2. How are the four quarter section corners of section 6 in any township located?

3. A rectangular city block was subdivided into 12 lots of uniform size 50'x120', making the block dimensions 300' x 250'. A resurvey to locate the corners of lot #3 revealed that the block dimensions were actually 304.8ft. x 247.5 feet. What should the dimensions of lot #3 be, and by what measurements should the corners thereof be determined?



4. For each of the above cases (given in problems 1, 2, 3) state briefly the functions of the surveyor, and indicate how those functions are limited by law and practice.



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